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BINGHAMTON WASTEWATER MANAGEMENT STUDY. DESIGN AND COST APPENDI--ETC(U)
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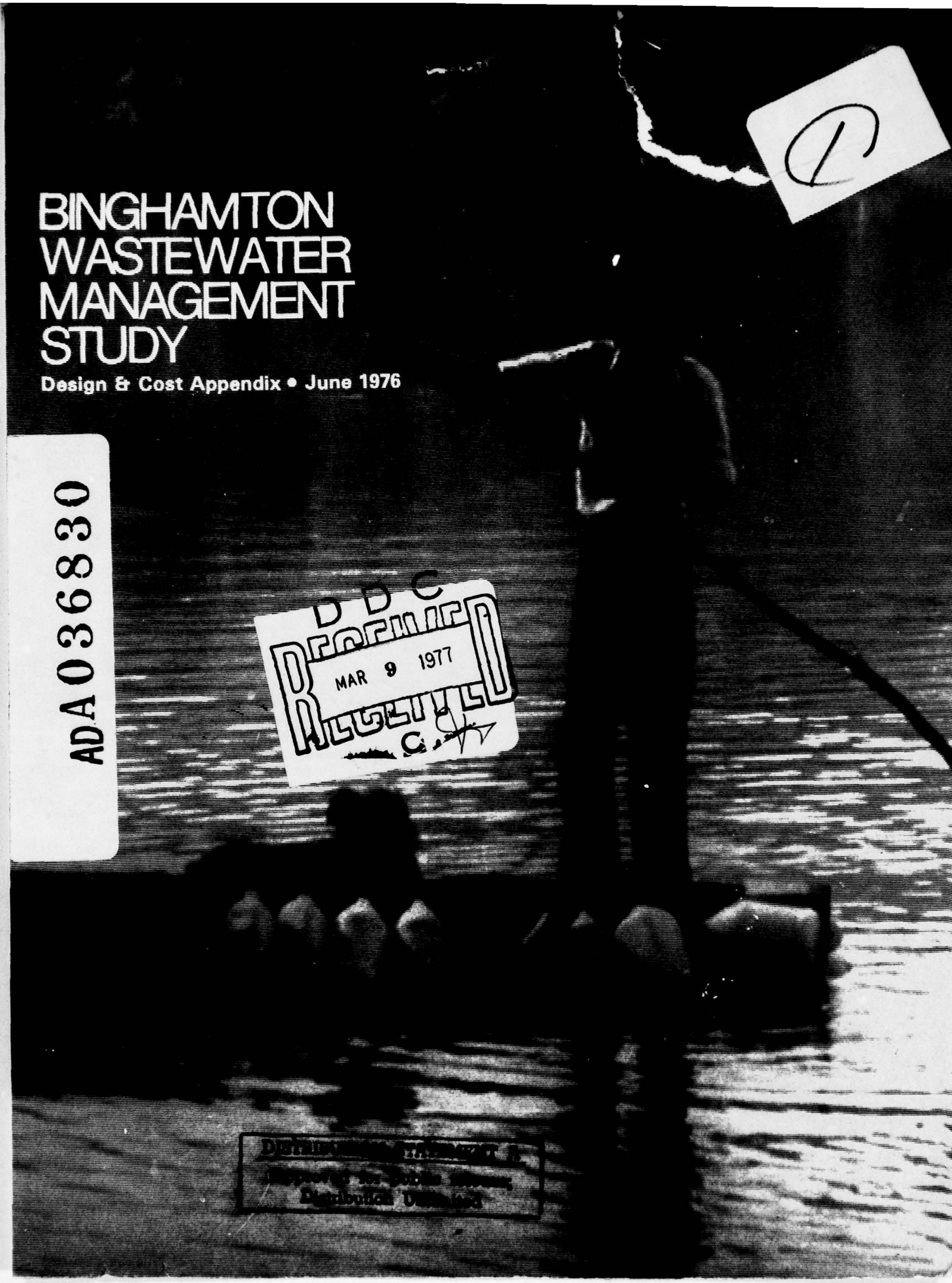
BINGHAMTON WASTEWATER MANAGEMENT STUDY

Design & Cost Appendix • June 1976

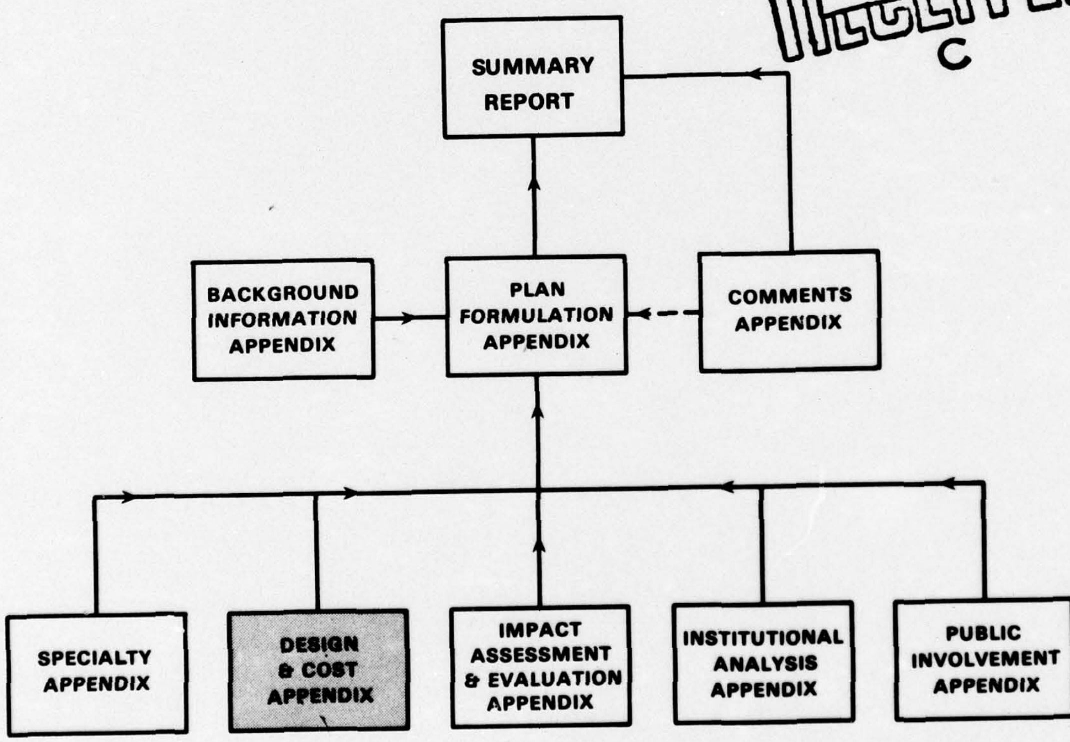


"The Raft of Summer"
Courtesy of Paul Smith, Binghamton, New York

Someone once said a picture is worth a thousand words, and the cover photograph summarizes the study in a simple but graphic manner. Today, the modern Huck Finn can enjoy many scenic and recreational opportunities associated with a Susquehanna River relatively free of pollutants. But tomorrow when the boy is grown, will the river still offer clean water for his children's enjoyment? This study suggests some ways to keep the Susquehanna clean and to ensure that future generations in Broome and Tioga Counties can enjoy "The Raft of Summer."

The Report for the Binghamton Wastewater Management Study consists of nine appendices. The Summary Report, the Background Information Appendix, the Plan Formulation Appendix, and the Comments Appendix constitute the primary Study documents. The five remaining documents support the Plan Formulation Appendix. The relationship of the Design and Cost Appendix to the other documents is indicated in the diagram below.

The Design and Cost Appendix documents the engineering analyses which were performed during the Study in arriving at design and cost data for various wastewater, stormwater and sludge management systems in Broome and Tioga Counties, New York. The final Plans for Choice are presented in complete detail in the last chapter.



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BINGHAMTON WASTEWATER MANAGEMENT STUDY,

DESIGN AND COST APPENDIX ,

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Prepared by the

BALTIMORE DISTRICT, CORPS OF ENGINEERS

DEPARTMENT OF THE ARMY

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BINGHAMTON WASTEWATER MANAGEMENT STUDY

DESIGN & COST APPENDIX

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CHAPTER I

INTRODUCTION TO THE STUDY AND THE REPORT

PREFACE

The Binghamton Wastewater Management Study was a cooperative Federal, State, and local planning effort to develop viable alternative plans for the protection and enhancement of water quality and associated resources of the Susquehanna River Basin within Broome and Tioga Counties, New York.

This appendix presents the documentation of the engineering analyses which formed the basis of the designs and costs of wastewater management systems. To a certain extent, the design and cost information was refined in an interative fashion similar to the refinement of the plans themselves. The Design and Cost Appendix presents the development and methodology of the engineering analyses as well as the detailed design and cost of the final plans. The Plan Formulation Appendix documents all significant events and decisions involved in the formulation, evaluation, and selection of wastewater management plans. Other supporting appendices, presenting detailed information on particular elements of the Study include the following: Impact Assessment and Evaluation, Institutional Analysis, and Public Involvement. Also included is the Speciality Appendix which presents detailed analyses of investigations that either overlapped the other appendices or were of particular interest to participants in the Study. These investigations were: the Outlying Communities, river recreation, nonstructural measures, land application, non-point source pollution, and industrial wastewater management. A general profile of the Study Area is contained in the Background Information Appendix.

The body of the Design and Cost Appendix is divided into eight sections:

The design criteria discussion (Chapter II) summarizes the population and wasteflow projections used as the framework for the technical design of alternatives. Additionally, the analyses performed in development of these projections are presented.

The economic analytical framework of the study is also presented in this chapter covering cost estimating procedures and elements of the cost-effectiveness analysis. In Chapter III, the existing wastewater management systems are described and the expected changes to be made by 1977 (Baseline Condition).

The methodology used in developing the components of the alternatives (Chapter IV) details the manner in which the various planning criteria were incorporated into the technical design. The development of the water quality model for the Susquehanna River is also presented.

Chapters V, VI, and VII detail the major technological design analyses for wastewater facility designs, sludge management, and stormwater management, respectively.

Design and cost of the four final plans is presented in Chapter VIII. An in-depth presentation is made of the design, construction schedule, costs, and performance features of each plan.

STUDY PURPOSE AND SCOPE

The purpose of the Binghamton Wastewater Management Study was to develop both short and long range wastewater management plans for the metropolitan area of Broome and Tioga Counties, New York. The plans were formulated to complement the water quality programs of the State of New York, achieve the objectives of the Federal Water Pollution Control Act Amendments of 1972 (Public Law 92-500), and meet the planning requirements of the Water Resources Council's "Principles and Standards for Planning Water and Related Land Resources."

LEVEL OF DETAIL

The level of design detail used in the Study was of "survey scope" detail. A survey scope investigation develops sufficient design and evaluation detail to allow a choice to be made among alternatives. It does not, however, provide detailed design data sufficient for the construction of a project. The level of detail achieved in a Corps of Engineers survey scope investigation is comparable to the detail developed in Areawide Waste Treatment Management Planning, Section 208 of P. L. 92-500, under the direction of the US Environmental Protection Agency (EPA).

The content of a 208 plan is described in the Federal Register dated 13 May 1974, 40 CFR 35, Subpart F, (Interim Regulations), and is summarized in Table I-1. Accomplished by the Binghamton Wastewater Management Study were items a, b, c, d, f, k, l, and n. Partially completed were items e, m, and q. Items g, h, and j did not apply to the Study Area, and item i was not investigated. Items o and p are State and local responsibilities.

TABLE I-1

REQUIREMENTS OF 208 AREAWIDE WASTE TREATMENT MANAGEMENT PLANNING

a. Identification of anticipated municipal and industrial treatment works construction over a 20-year period.

b. The plan shall include those portions of facilities planning in compliance with Section 201, Step 1, (Table I-2), and for which Step 2 or Step 3 (Section 201), grant assistance is expected during the 5-year period following Section 208 plan approval.

c. Identification of required urban stormwater runoff control systems.

d. Establishment of construction priorities over 5- and 20-year periods.

e. Establishment of a regulatory program to (1) identify and control all point and nonpoint pollution sources; (2) regulate the location modification and construction of waste-discharging facilities; and (3) assure that industrial or commercial wastes discharged into publicly-owned treatment works meet applicable pretreatment requirements.

f. Identification of agencies necessary to construct, operate, and maintain facilities required by the plan.

g-j. Identification of nonpoint sources of pollution related to agriculture and silviculture (g), mining (h), construction (i), and saltwater intrusion (j); and procedures and methods (including land use requirements) to control those sources to the extent feasible.

k-ℓ. Processes to control the disposition of residual waste (k) and land disposal of pollutants (ℓ) to protect ground and surface water quality.

m. Selection of a management agency(s) and institutional arrangement to implement the plan and identification of the major management alternatives.

n. A schedule for implementing all elements of the plan, including identification of the monetary costs and economic, social, and environmental impact of implementation.

o-p. Required certification relating to consistency with other plans (o), and to public participation (p) in the planning process and plan adoption.

q. Recommendation of a plan by appropriate local governing bodies for state certification and EPA approval.

Also accomplished by this Study are certain elements of Step 1, Section 201 of P. L. 92-500, Wastewater Management Facilities Planning. The goal of Step 1 facilities planning is to formulate and analyze the most cost-effective and environmentally effective waste management alternatives for proposed waste treatment construction projects. Such planning then governs investment decisions for municipal facilities within each community. Facilities planning, or Step 1, is part of an integrated construction grants process, and is generally completed before Step 2 (construction drawings and specifications) grants, or Step 3 (actual construction) grants are awarded.

The content of a 201 Plan is described in the Federal Register dated 11 February 1974, 40 CFR 35, Subchapter B, (Final Regulations), and is summarized in Table I-2. Accomplished by this Study were items a, b, d, e, g, h, and i. Portions of items c and f were also accomplished.

INTEREST RATE

For preliminary cost estimation, an interest rate of 6 percent was used. In later, more refined work, the interest rate used in developing costs was that mandated by the Water Resources Council. The rate in effect throughout most of the Study was 5 7/8 percent. Near Study completion, however, the interest rate for Federal water resources plans was revised upwards to 6 1/8 percent. All costs for the final four plans reflect this new rate.

PERIOD OF ANALYSIS

The planning period selected for the Study was 1977 through 2020. Formulating, assessing, and evaluating all plans through the year 2020 assured the consideration of long-term environmental effects. For purposes of cost analyses, a

TABLE I-2

201 FACILITIES PLAN REQUIREMENTS

a. A description of the treatment works for which construction drawings and specifications are to be prepared.

b. A description of the selected complete waste treatment system of which the proposed treatment works is a part.

c. Infiltration/inflow documentation in accordance with Section 35.927.

d. A cost-effectiveness analysis of alternatives to include:

(1) relationship of size and capacity of alternative works to the needs to be served;

(2) evaluation of alternative flow and waste reduction measures;

(3) evaluation of improved effluent quality attainable by upgrading the operation and maintenance and efficiency of existing facilities;

(4) evaluation of the capability of each alternative to meet applicable effluent limitations;

(5) Identification of, and provision for, applying the best practicable waste treatment technology.

(6) Evaluation of alternative means by which ultimate disposal can be effected for treated wastewater and for sludge materials.

(7) Assessment of environmental impacts.

e. Identification of effluent limitations, or a copy of the NPDES permit if one has been issued.

f. Required comments or approvals of relevant State, interstate, regional and local agencies.

g. Summary of any public meetings or hearings held during the planning process.

h. Statement of how the plan will be implemented within the time period proposed.

i. Statement specifying that Title VI requirements of the Civil Rights Act of 1964 havebeen satisfied and that Part 7 has been satisfied.

50-year (1977--2027) project economic life was used. In accordance with EPA guidelines, a 20-year (1977--1996) economic life was also used for the final plans. This short-term analysis is presented in Addendum A.

PRODUCT OF THE STUDY

The product of the Binghamton Wastewater Management Study was an areawide investigation of wastewater management problems and solutions in Broome and Tioga Counties, New York. In this regard, the product of the Study was comparable to the planning activities accomplished under EPA's "208" programs. The Binghamton Wastewater Management Study fulfills most of the requirements of the EPA 208 program while satisfying the Corps' guidelines for survey scope reports.

The survey-scope study then, formulated and analyzed cost-effective and environmentally-effective wastewater management alternatives. These alternatives ranged from a plan to meet current requirements within the immediate future to a plan to achieve the highest levels of wastewater treatment by the year 1985, consistent with the goals of P. L. 92-500. Additionally, the alternatives were designed to the year 2020 with an accompanying schedule of construction priorities and implementation procedures. The final plans are displayed in this Report (Chapter VIII) as an aid for decisions on an areawide basis.

The final plans have been presented to State and local officials for their selection of a recommended plan. The adopted areawide plan will form a basis for applying to EPA for grants in connection with the 201 facilities program for individual projects. As previously mentioned, some of the advanced planning normally accomplished under the 201 Step 1 activity has already been performed in this Study.

The adopted plan can also be used to evaluate and, if necessary, amend the current wastewater comprehensive plans for each county. Whichever course of action the local and State decision-makers choose, the Report is intended as an aid to Broome and Tioga Counties in future planning for wastewater management.

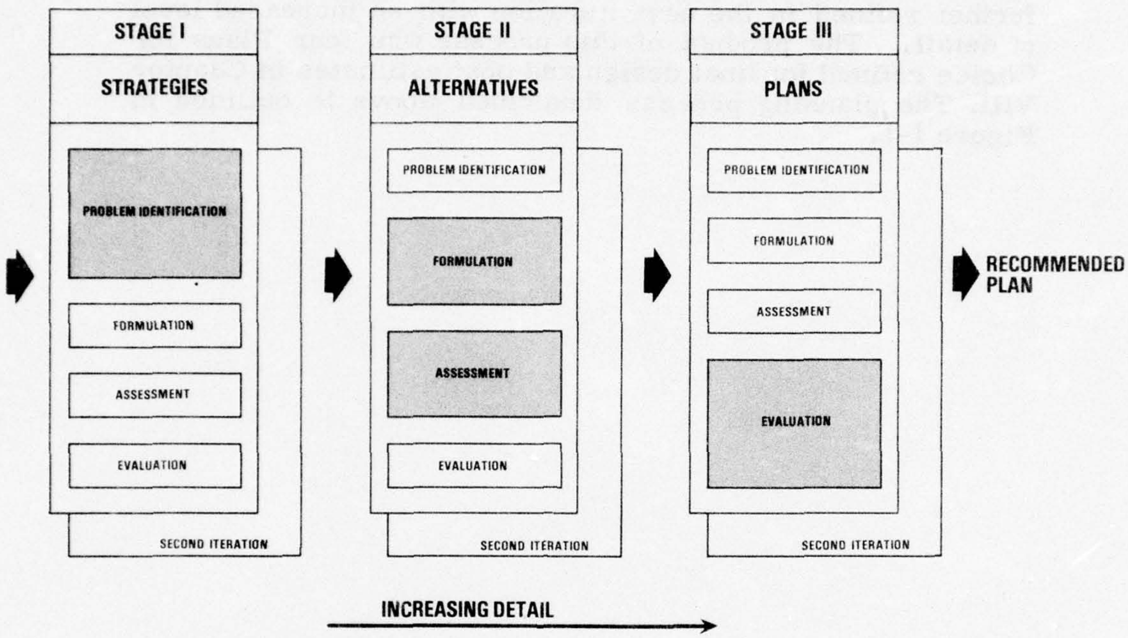
The Corps of Engineers role in this effort was limited to providing a wastewater management planning service for local and State governments. The Corps of Engineers has no authority for construction of wastewater treatment facilities.

PLANNING PROCESS

In developing the plans, a sequential process of three stages was used. In Stage I, a broad range of strategies were delineated with few restrictions. Technical performance criteria for a wide range of possible systems were investigated. Costs were based on national averages for typical systems and not on specific Study Area factors.

In Stage II, specific wastewater management alternatives were developed. Also, four storm overflow control alternatives for the City of Binghamton's combined sewer system and three alternative sludge management methods for all service areas were investigated. Detailed costs were developed to allow selection of the best combinations of treatment components. The technical design was accomplished by an in-depth consideration of the existing and proposed treatment sites and local conditions requiring special analysis. These steps represented a major increase in the level of detail provided during Stage I. Rather than relying on generalized cost curves used in Stage I, cost-estimation procedures incorporated local conditions affecting the construction and operation of wastewater facilities. Where the screening of alternatives was noted to be sensitive to critical assumptions in the technical

design, cost-estimation, or impact assessment, further detail and refinement was provided. Complete treatment systems were formed in Stage III with the components refined where necessary to be comparible with the other elements and in meeting the objectives of the plan in the least cost way. In each of the three stages, several iterations were performed. Each iteration worked on the successive tasks of problem identification, technical design and cost, impact assessment, and evaluation. After each iteration the selected alternatives, or components of alternatives, were further refined in the next iteration with an increased level of detail. The product of this process was four Plans for Choice refined for final design and cost estimates in Chapter VIII. The planning process described above is outlined in Figure I-1.



Relationship of Planning Stages, Tasks, and Iterations

FIGURE I-1

CHAPTER II

DESIGN AND COST CRITERIA

This chapter discusses items which constituted the general analytical framework for the design and cost of wastewater management systems. These items include population and flow projections for each service area and the cost methodology used for developing both the preliminary and the detailed alternatives.

STUDY AREA

For planning purposes, 18 wastewater management areas were designated in Broome and Tioga Counties based on existing and projected development. Wastewater management areas do not necessarily conform to municipal boundaries, but rather include areas which could reasonably be incorporated into a single management district. These areas were divided into urban and outlying regions as shown in Figure II-1. The latter category is made up of the nine Outlying Communities and is discussed in the Speciality Appendix. Plans were not developed in detail for these areas. The nine urban wastewater management areas constituted the Urban Study Area, and were the focus of the Study. These areas were Binghamton-Johnson City, Endicott, Vestal, Nanticoke Valley, Chenango Valley, and Five Mile Point in Broome County; and East Owego, West Owego, and Owego Village in Tioga County. These areas are described in Table II-1. Flows from Five Mile Point will be sent to

BICOUNTY STUDY AREA

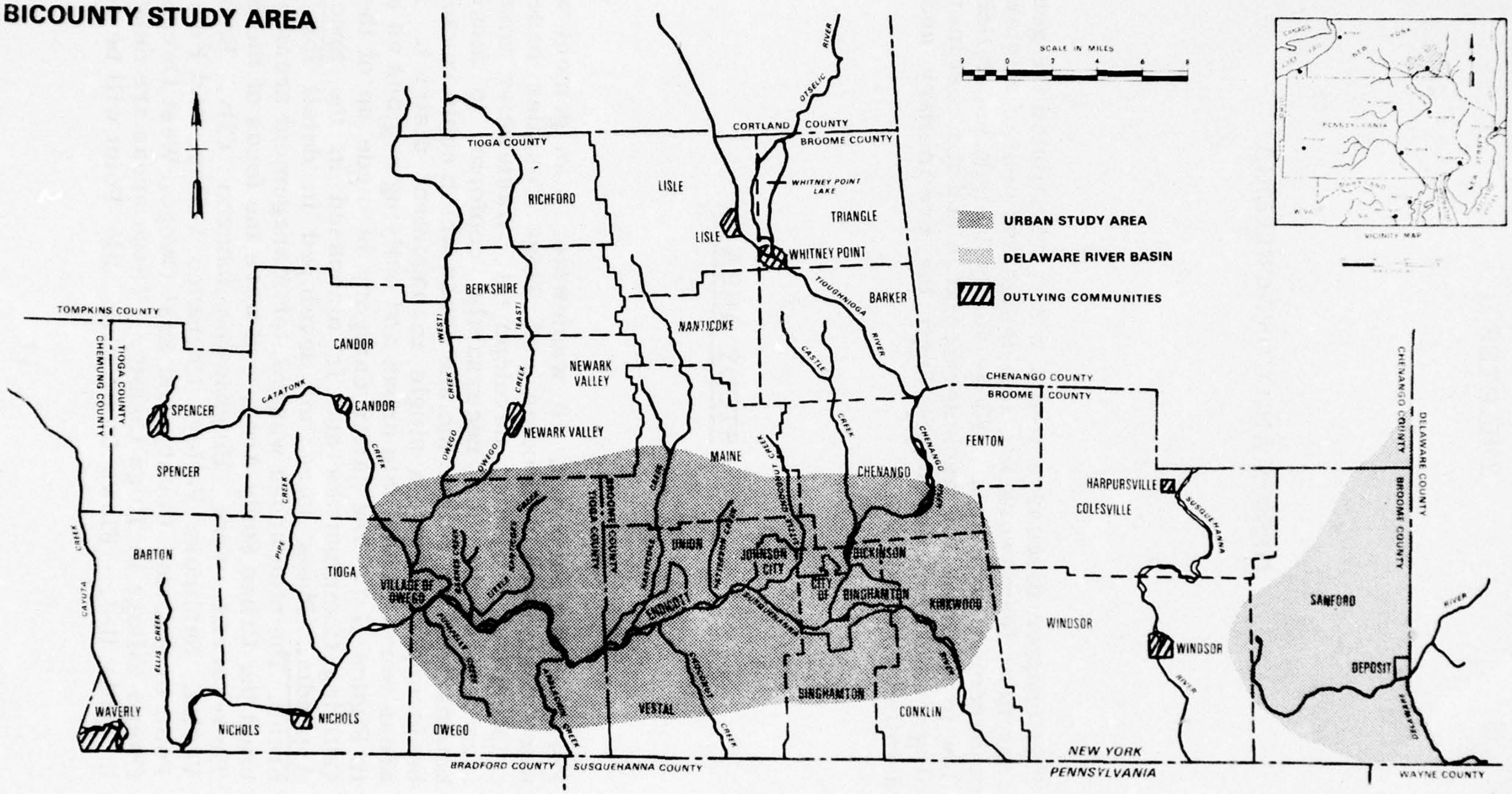


Figure II-1

TABLE II-1

CHARACTERISTICS OF WASTEWATER MANAGEMENT AREAS

Wastewater Management Area	Existing Sewers	Comments
STUDY AREA		
1. Binghamton-Johnson City	Yes	Served by secondary sewage treatment plant (STP). Proposed sewer extensions to area 7 and 8. Combined sewer overflows and infiltration problems. Some industrial discharges. Both separate and combined sewers.
2. Endicott	Yes	Secondary STP. Some sewer expansion in Union. Sewers partially combined. Some industrial discharge.
3. Vestal	Yes	Primary STP to be abandoned. Planned interceptor for West Vestal to Endicott. East Vestal served by Binghamton-Johnson City STP. Separate storm sewers. Planned expansion of service area in West Vestal.
4. East Owego	Yes	Secondary treatment at Town of Owego STP No. 2. Separate sewers but some infiltration/inflow problems.
5. West Owego	Yes	Secondary treatment at Town of Owego STP No. 1. Both separate and combined sewers. Some industrial discharges.
6. Owego Village		Plans to upgrade primary plant to secondary. Sewers partially combined. Infiltration problems. Valley View Imhoff tank to be abandoned.
7. Nanticoke Valley	No	Proposed future growth area and planned sewer hookup with Endicott STP.
8. Chenango Valley	No	Planned sewering. Connection to either Binghamton-Johnson City STP or separate Chenango Valley STP.
9. Five Mile Point	Yes	Some sewering with additional extensions planned to be hooked up to Binghamton-Johnson City STP.

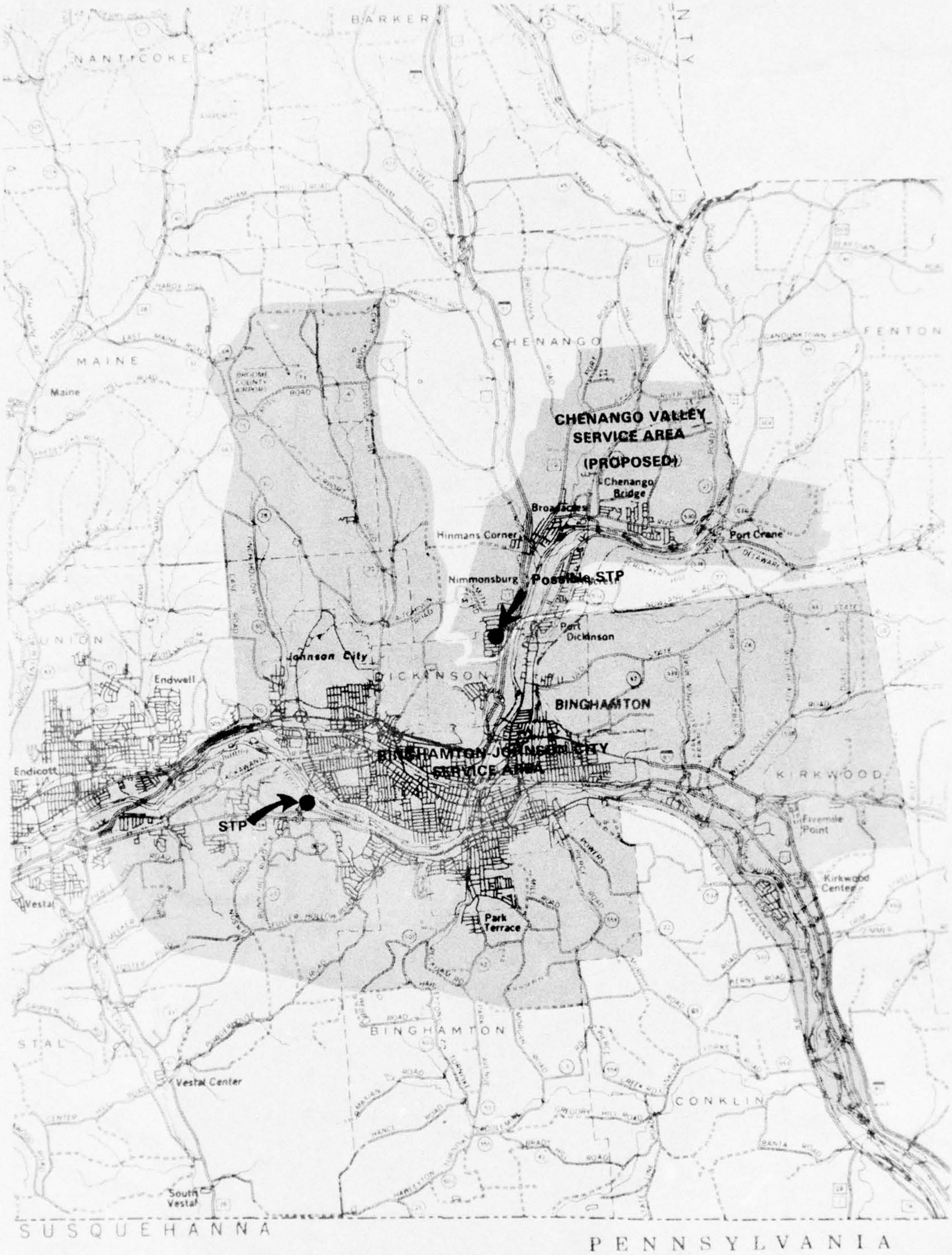
the Binghamton-Johnson City STP. Vestal and Nanticoke Valley wastewater is expected to be treated at the Endicott STP by 1977. Thus, there were six possible STP service areas with all except Chenango Valley serviced by existing STP's. The service areas for Binghamton-Johnson City and Chenango Valley are shown in Figure II-2. The Endicott service area is depicted in Figure II-3, and the Owego service areas are shown in Figure II-4. Table II-2 lists the major place names of each of these services areas. Two approaches were considered in providing sewerage to Chenango Valley: (1) an initially limited service area (Phase I) with expansion to full service after a period of about years; and (2) full sewer service throughout the Valley at the beginning of the planning period (1977). The Phase I service area and and total service area are shown in Figure II-5.

POPULATION PROJECTONS

DERIVATION

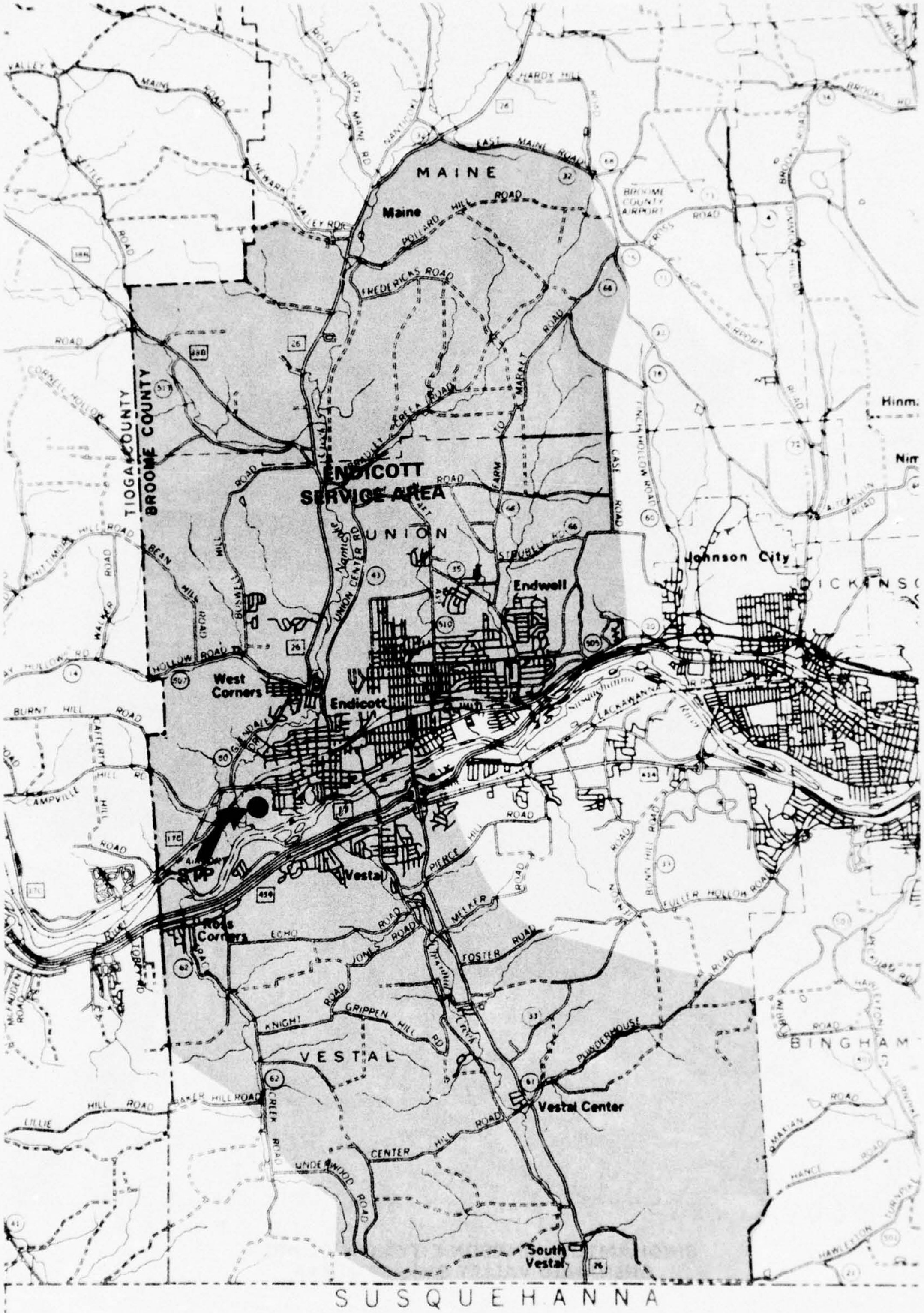
Population trends within the urban areas indicated increasing urbanization adjacent to older and larger communities. The City of Binghamton, Johnson City, Endicott, and the Village of Owego have lost population since 1950, while many of the small municipalities within the Binghamton Metropolitan area have experienced significant growth.

Municipal population projections were developed on a town and village basis (see Background Information or Plan Formulation Appendices). As the Study Area limits did not always coincide with municipal boundaries, the population projections were further broken down into submunicipal areas based on sewer, political, hydrologic, and topographic boundaries. These submunicipal areas are outlined in Figure II-6 and described in Table II-3. The populations projections for each sub-area are tabulated in Table II-4 and were disaggregated from the municipal total using the ratio method (percent land of sub-area to total municipal area) as a general guide. Adjustments were made for factors such as topography, access, vacant land, and availability of commercial areas, schools, and other support facilities.



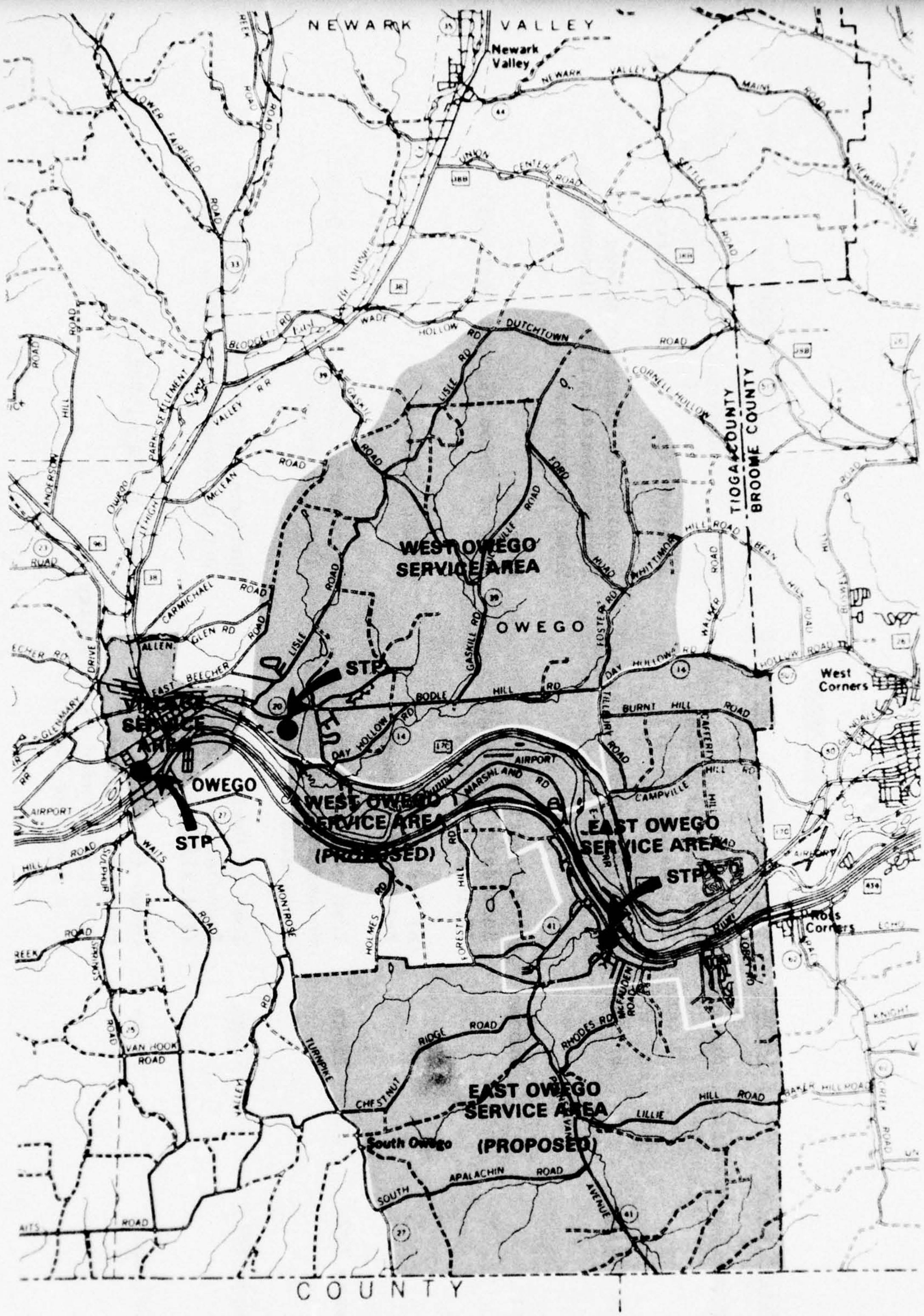
BINGHAMTON-JOHNSON CITY SERVICE AREA
CHENANGO VALLEY SERVICE AREAS

FIGURE II-2



ENDICOTT SERVICE AREA

FIGURE II-3



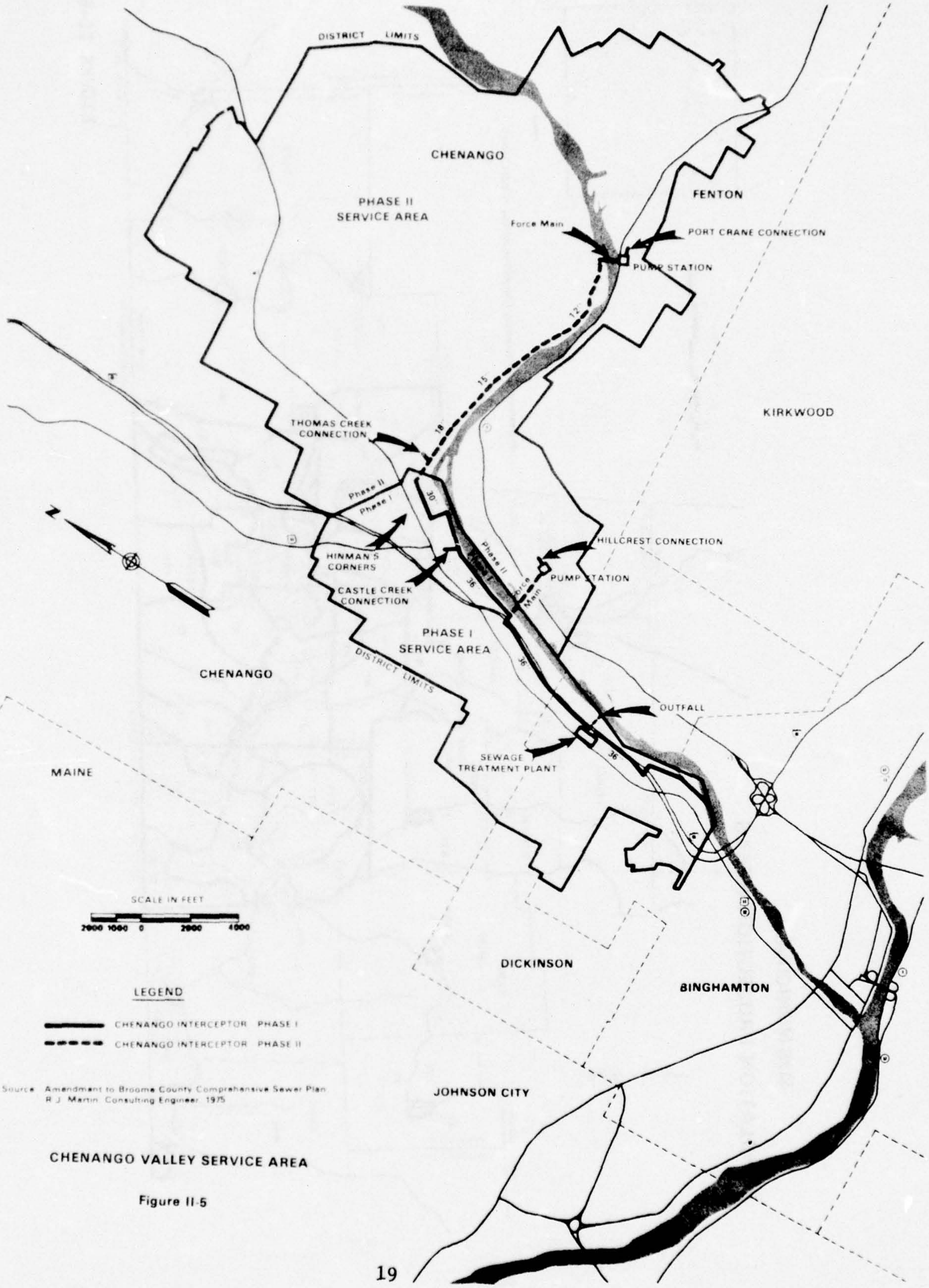
OWEGO SERVICE AREAS

FIGURE II-4

TABLE II-2

SERVICE AREAS

STP SERVICE AREA	COUNTY	EXISTING SEWAGE TREATMENT	MAJOR PLACE NAMES INCLUDED
Binghamton-Johnson City	Broome	B-JC STP Secondary Treatment (Activated Sludge)	City of Binghamton, Town of Binghamton Kirkwood, Port Dickinson, Dickinson Johnson City, Westover, Portions of Vestal
Chenango Valley	Broome	None (No sewers existing)	Broadacres, Chenango Bridge Fenton--Kattellville, Nimmonsburg Sunrise Terrace
Endicott	Broome	(a) Endicott STP Secondary Treatment (Trickling Filter)	Endicott Town of Union (Except Westover)
		(b) Vestal STP Primary Treatment	Most of Vestal
East Owego (#2)	Tioga	Town of Owego STP #2 Secondary Treatment (Activated Sludge)	Eastern Portions of Town of Owego (See Figure II-4)
West Owego (#1)	Tioga	Town of Owego STP #1 Secondary Treatment (Trickling Filter)	Northern and Western portions of Town of Owego (See Figure II-4)
Owego Village	Tioga	Owego (V) STP Primary Treatment (To be upgraded to secondary in Baseline Plan.)	Owego Village, Valley View



Source: Amendment to Broome County Comprehensive Sewer Plan, R.J. Martin, Consulting Engineer, 1975

CHENANGO VALLEY SERVICE AREA

Figure II-5

SUB-MUNICIPAL
POPULATION PROJECTION AREAS

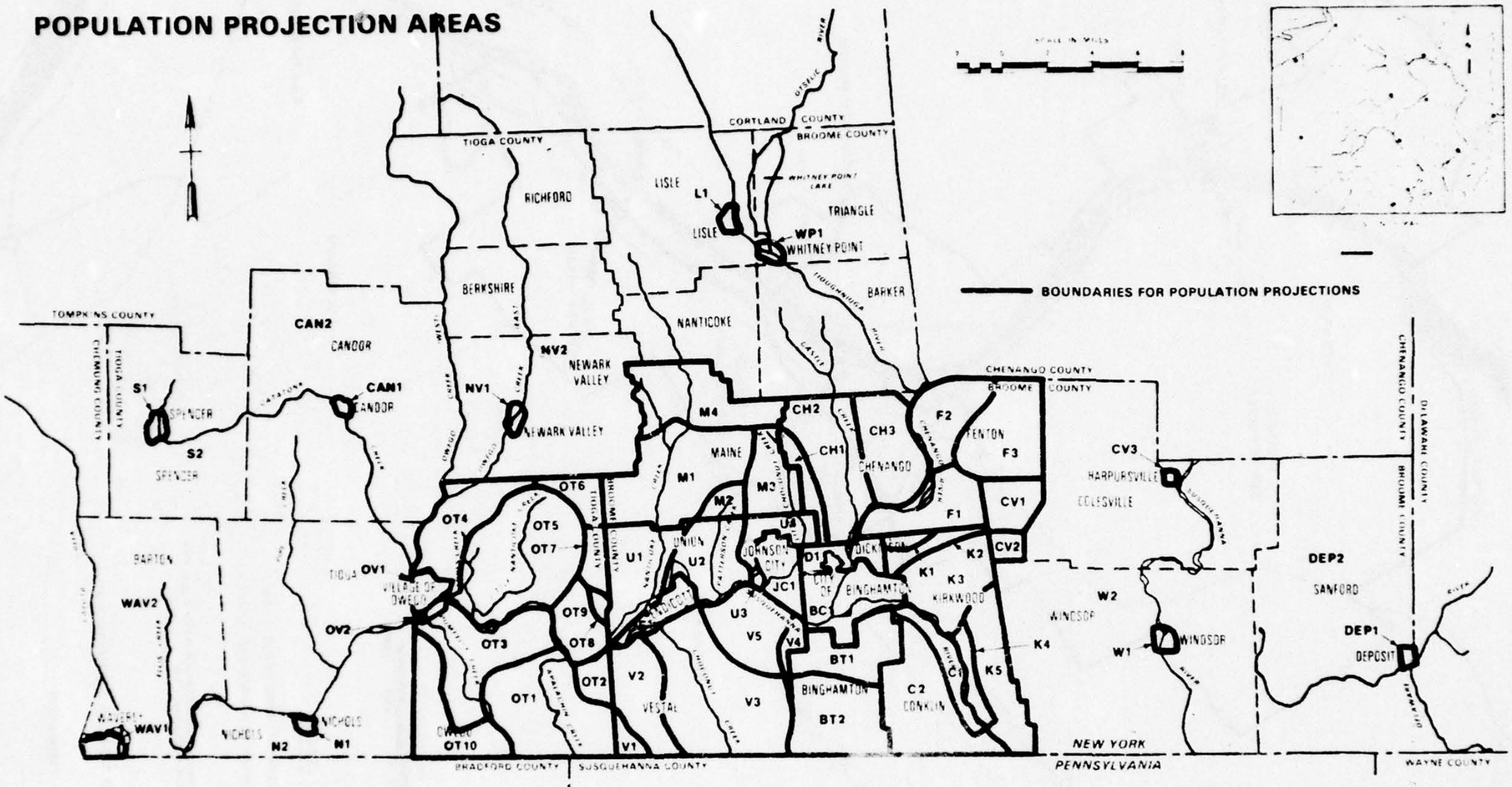


FIGURE II-6

TABLE II-3

SUB-MUNICIPAL POPULATION PROJECTION AREAS

MUNICIPALITY	SUB-MUNICIPAL DESIGNATION (See Figure II-6)	DESCRIPTION
Broome County		
Binghamton City	BC-1	Entire City
Binghamton Town	BT-1	Northern half of town drained by Pierce and Boyle Creeks
	BT-2*	Southern half of town
Chenango	CH-1	Western part of town drained by Little Choconut Creek
	CH-2	Middle part of town, drained by Castle Creek
	CH-3	Eastern part of town, drained by Gilbert Creek
Conklin	C-1	Urbanized area along river
	C-2	Remainder of town
Dickinson	D-1	Entire town
Fenton	F-1	Southern portion of town
	F-2	Northern portion of town
	F-3*	Eastern portion of town
Kirkwood	K-1*	Phelps Creek Basin
	K-2*	Brandywine Creek area
	K-3	General area drained by Stratton Mill, Stanley Hollow, and Acrea Creeks. Includes the industrial park.
	K-4	Southern area along river
	K-5	Rural, eastern part of town

*Not included in Study Area.

TABLE II-3 (Con't)

MUNICIPALITY	SUB-MUNICIPAL DESIGNATION	DESCRIPTION
Broome County (Cont'd)		
Maine	M-1	Nanticoke Creek Basin to Maine Center
	M-2	Patterson Creek Basin
	M-3	Little Choconut Creek Basin
	M-4	Northern, rural areas
Endicott	E-1	Entire village
Johnson City	JC-1	Entire village
Union	U-1	Nanticoke Creek Basin
	U-2	Patterson Creek Basin
	U-3	Westover
	U-4	Little Choconut Creek
Vestal	V-1*	Appalachin Creek Basin
	V-2	Tracy Creek Basin
	V-3	Choconut Creek Basin (except area V-5)
	V-4	Eastern part of town in Fuller Hollow Creek Basin
	V-5	Portion of Choconut Creek Basin presently served by Binghamton-Johnson City STP
Tioga County		
Owego Village	OV-1	North of Susquehanna River
	OV-2	South of Susquehanna River

*Not included in Study Area.

TABLE II-3 (Con't)

MUNICIPALITY	SUB-MUNICIPAL DESIGNATION	DESCRIPTION
Owego Town	OT-1	Appalachin Creek Basin
	OT-2	Tracy Creek Basin
	OT-3	Pumpelly Creek Basin
	OT-4	Owego Creek Basin
	OT-5	Little Nanticoke Creek Basin
	OT-6*	Crocker Creek Basin
	OT-7*	Day Hollow Creek Basin
	OT-8*	Dead Creek Basin
	OT-9	Crestview Heights - Campville
	OT-10 *	Portion draining to Pennsylvania and Hunts Creek.

*Not included in Study Area.

TABLE II-4

POPULATION PROJECTIONS FOR SUB-MUNICIPAL AREAS

Municipality	Sub-Area	1973	1980	1990	2000	2010	2020
Broome County							
Binghamton City	BC-1	62,820	60,200	57,900	57,900	58,400	60,000
Binghamton Town	BT-1	3,780	4,000	4,570	4,700	4,850	5,150
	*BT-2	1,390	1,500	1,650	1,700	1,750	1,850
	Total	5,170	5,500	6,220	6,400	6,600	7,000
Chenango	CH-1	410	420	450	480	500	530
	CH-2	5,840	5,850	6,250	6,420	6,600	6,720
	CH-3	6,780	6,800	7,200	7,400	7,600	7,750
	Total	13,030	13,070	13,900	14,300	14,700	15,000
Conklin	C-1	4,540	4,540	4,850	5,150	5,350	5,500
	C-2	1,210	1,210	1,270	1,350	1,450	1,600
	Total	5,750	5,750	6,120	6,500	6,800	7,100
Dickinson	D-1	5,690	5,370	5,150	5,100	5,200	5,300
Fenton	F-1	4,740	4,850	4,950	5,500	5,900	6,200
	F-2	1,490	1,600	1,720	2,300	2,550	3,000
	*F-3	530	570	630	900	950	1,000
	Total	6,760	7,020	7,300	8,700	9,400	10,200

*Not included in Study Area

TABLE II-4 (Con't)

Municipality	Sub-Area	1973	1980	1990	2000	2010	2020
Kirkwood	*K-1	110	110	120	130	150	160
	*K-2	240	240	250	260	300	320
	K-3	3,100	3,110	3,330	3,400	3,500	3,600
	K-4	1,580	1,590	1,700	1,800	1,900	2,050
	K-5	1,050	1,050	1,100	1,110	1,150	1,170
	Total	6,080	6,100	6,500	6,700	7,000	7,300
Maine	M-1	2,860	3,480	3,920	4,070	4,240	4,500
	M-2	290	330	400	430	450	500
	M-3	1,110	1,140	1,200	1,250	1,300	1,350
	M-4	1,600	1,820	2,350	2,450	2,500	2,650
	Total	5,860	6,770	7,900	8,200	8,500	9,000
Endicott	E-1	16,320	15,800	15,200	15,200	15,400	15,600
Johnson City	JC-1	17,750	17,400	16,880	16,900	17,100	17,300
Union	U-1	9,030	9,330	9,550	9,550	9,600	9,650
	U-2	19,360	19,800	20,230	20,250	20,350	20,400
	U-3	750	750	750	750	750	750
	U-4	860	900	950	950	1,000	1,000
	Total	30,000	30,750	31,500	31,500	31,700	31,800

*Not included in Study Area.

TABLE II-4 (Con't)

Municipality	Sub-Area	1973	1980	1990	2000	2010	2020
Vestal	*V-1	40	60	80	80	80	80
	V-2	1,710	2,080	2,500	2,600	2,620	2,660
	V-3	13,300	15,040	18,340	18,900	18,970	19,190
	V-4	6,960	8,395	9,600	10,200	10,580	10,720
	V-5	4,200	6,000	6,800	7,600	7,600	7,700
	Total	26,210	31,575	37,320	39,380	39,850	40,350
Tioga County							
Owego Village	OV-1	4,680	4,390	4,190	4,140	4,340	4,540
	OV-2	460	460	460	460	460	460
	Total	5,140	4,850	4,650	4,600	4,800	5,000
Owego Town	OT-1		4,300	7,000	7,400	7,670	8,000
	OT-2	4,200	5,230	6,000	6,300	7,100	7,300
	OT-3	1,500	1,730	2,610	2,800	2,840	3,100
	OT-4	960	1,270	1,300	1,350	1,400	1,450
	OT-5	2,620	2,800	4,400	4,800	5,070	5,400
	*OT-6	190	230	310	340	370	400
	*OT-7	180	230	310	340	370	400
	*OT-8	130	160	220	250	270	300
	OT-9	3,380	4,000	5,050	5,230	5,720	6,000
	*OT-10	480	550	600	650	700	750
	Total	17,600	20,500	27,800	29,400	31,510	33,100

*Not included in Study Area.

Table II-4 also presents population estimates for 1973. These estimates are derived from various techniques. In many cases, estimates were based on house counts within each sub-area from 1973 Planimetric maps and from water connection records and water district maps. Census data were used to compute the average number of persons per household. In areas where a significant number of multi-family dwellings was noted, a correction factor derived from census estimates was applied to the analysis. The 1973 population estimates were then obtained by multiplying the house counts for 1973 by the average number of persons per household (assumed to have remained constant since the census).

Several of the submunicipal areas have a low density of present and expected future development and for this reason were not included in the Urban Study Area (indicated by an asterisk in Table II-3). Those included in the Urban Study Area were aggregated into the treatment plant service areas previously discussed. The submunicipal areas included in each service area are listed in Table II-5.

REVISIONS

The population projections developed early in the Study were modified in later work as follows:

1. Town of Dickinson. The northern portion of the Town of Dickinson was added to the Chenango Valley service area, as shown in Figures II-1 and II-5, which resulted in a population of about 400 plus the Broome Community College being removed from the Binghamton-Johnson City service area. Since Dickinson is not expected to experience any population growth, the same adjustment was made to the Chenango Valley service area for the population projections.

2. Towns of Chenango and Fenton. Sewering in the Chenango Valley was limited to the county sewer district limits as shown in Figure II-5. All people within the overall district limits were included in the population estimate of the Chenango Valley service area. In order to derive projections of future populations for the district, the assumption was made that all new population growth projected in submunicipal areas CH2, CH2.2, F1, and F2 would occur within the existing district limits, or adjacent to the district such that-district extensions could be readily created.

TABLE II-5

RELATIONSHIP OF SERVICE AREAS TO SUBMUNICIPAL AREAS

Service Area	Submunicipal Areas*
Binghamton-Johnson City	BC1, K3, BT1, V4, V5, U3, U4, JC1, M3, CH1, D1, C1, C2, K4, K5
Chenango Valley	CH2, CH3, F1, F2
Endicott	E1, U1, U2, V2, V3, M1, M2, M4
Owego Village	OV1, OV2
East Owego	OT9, OT2, OT1
West Owego	OT3, OT4, OT5

*Defined in Table II-3.

3. West Owego Service Area. The population projections in this area assume: (a) population growth in projection area OT5 will occur in or adjacent to the existing sewer districts such that this new population can be readily sewered; and (b) area OT3 is considered to be divided into two areas--that draining via Pumpelly Creek to Owego Village, and that area not draining to the Village. Both areas are assumed to experience population growth in proportion to the 1973 population estimates. All new growth in the area not draining to Owego Village is assumed to occur in the level land area along the River, which is projected to be sewered and connected to the West Owego plant after the year 1990.

The above refinements were made during Stage III and represented a major adjustment to the population projections used in earlier stages which relied on simple aggregation of the population projection areas. The change was made because the earlier projections assumed 100 percent sewering (i.e., all projected population would be sewered) of areas OT3, OT4, and OT5 in West Owego, and CH2, CH3, F1, and F2 in Chenango Valley. In the Binghamton-Johnson City, Endicott, and East Owego service areas, the proportion of the total population which is unsewered is quite small. Therefore, the initial assumption of 100 percent sewering was not modified in these areas because the somewhat optimistic projection had negligible impact on the total population projections. In the Chenango Valley and West Owego areas, the opposite (a larger proportion of unsewered population) was true, so the 100 percent sewering assumption was modified.

This assumption was modified to better estimate the capacities required for the various treatment plants. Additionally, the cost-effectiveness of treatment plant logistics changed with the modified projections. High population projections imply high wasteflow projections. The economics of regionalization show that the tendency of decision-making is to regionalize areas generating large wasteflows, and, conversely, to serve areas with separate plants if the flows are low. For example, the higher initial population projections (resulting from the 100 percent sewering assumption) for the West Owego service area resulted in the regionalization of West Owego and Owego Village appearing economically advantageous for secondary treatment of wastes. With the reduced projections selected for the Study, subregionalization had the least cost at the secondary treatment level. The revised population projections are presented in Table II-6.

TABLE II-6

STAGE III POPULATION PROJECTIONS
(Sewered Population)

Service Area	Population (in 1000's) 1973	1980	2000	2020
Binghamton-Johnson City	102.0	107.7	109.2	113.4
Five Mile Point[1]	---	8.4	9.5	10.4
TOTAL Service Area	---	116.1	118.7	123.8
Endicott	44.7	45.0	45.1	45.7
Vestal[2]	7.5	17.0	21.5	21.8
Nanticoke Valley[2]	---	5.6	7.0	7.7
TOTAL Service Area	---	67.6	73.6	75.2
Chenango Valley[3]	---	16.0	19.0	21.0
East Owego	6.5	13.5	18.9	21.3
West Owego	0.6	0.8	3.4	4.4
Owego Village[4]	3.6	4.9	4.6	5.0
Owego Valley View	0.5	---	---	---
TOTAL URBAN STUDY AREA	165.4	218.9	238.2	250.7

1. Included in Binghamton-Johnson City Service Area by 1977.
2. Included in Endicott Service Area by 1977.
3. Estimated 1973 population for Phase I Service Area is 8,000.
4. Includes population in Owego Valley View by 1977.

The plans (in Stage III-2 only) which provided phased sewer service to Chenango Valley result in a correspondingly smaller service population. The limited service area, shown in Figure II-5 as Service Area 1, includes part of Dickinson and Chenango Towns. Since the service limitation (Interim Plan) is only expected to last five years, and the area is almost completely developed, the population growth should be quite small relative to the existing population. The 1973 population estimate for the limited service area is 8,000 persons.

WASTEWATER FLOW PROJECTIONS

Two sets of wastewater flow projections were derived for this study: (1) the expected flows with no flow reduction measures, and (2) a low flow projection resulting from the implementation of nonstructural flow reduction measures (water pricing and public education). In the design of the final plans, only AWT systems were based on the low flow projections. The other final plans did not assume any reduction in flows due to nonstructural means as these measures were not cost-effective for treatment levels below AWT. With the exception of the Baseline, however, all plans did assume the cost-effective level of infiltration control for the City of Binghamton.

Projections of wastewater discharges to municipal sewers were based on existing and projected population and per capita flow rates. Per capita rates included commercial and industrial flows as well as residential flows. This approach was used because the amount of industrial process water discharged to municipal systems is not large. Most of the industrial wastewater represents employee domestic use. Existing infiltration flows were also included. This section will discuss the assumptions inherent in the projections, the derivation of the expected flow projections, and a summary of the analysis of the nonstructural flow reduction measures.

PER CAPITA FLOW RATES

The wastewater flow projections were primarily derived by multiplying the population of a given area by the existing or estimated per capita flow for that area. No definitive statement can be made as to the composition of the discharges to the local sanitary sewer systems. Sewage flows include infiltration into sewers, inflow from stormwater runoff and direct discharges from various classes of users. Although water supply may be metered to each user, as is generally the case in the Binghamton region, consumptive uses make the determination of existing discharges subject to additional error. The estimates and projections of discharge rates were based on a combination of water supply records, sewage treatment plant flow records, and engineering experience and judgment.

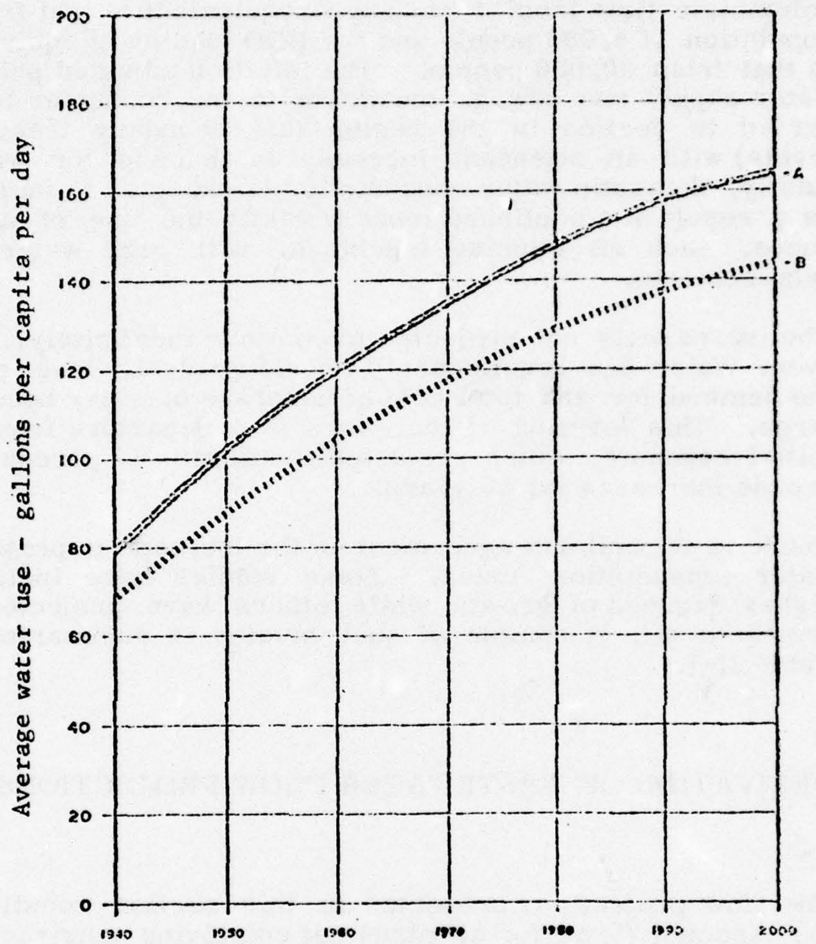
Residential water use in the Study Area was determined from 29 water districts (15 sewered and 14 unsewered), outside of Binghamton, Johnson City, and Endicott. Records in these three urban areas were also examined, but were generally not available in a compiled form suitable for analysis. Average per capita residential water use was 70 gallons per day in the unsewered districts and 67 gallons per day in the sewered districts. Assuming 90 percent of the water used is discharged as wastewater, the average per capita wastewater flow would be 60 gpcd for residential areas. This compares favorably with the per capita wastewater flow of 57 gpcd to the Town of Owego STP #2 which services a residential area (East Owego).

For areas projected to have a mix of industrial, commercial, and residential flows, an average per capita rate of 135 gpcd was used based on STP records for Vestal and West Owego. For the Urban Study Area, the per capita wastewater flow rates were expected to increase 10 percent during the 1970's, 10 percent during the 1980's, 5 percent during the 1990's, and no increase from 2000 to 2020. The overall increase from existing (1973) levels would be 27 percent. The increase is assumed to occur equally for areas of homogenous residential development as well as areas with mixed commercial/industrial activity. These increases are comparable to those developed by Sweitzer and Flentje (1972) in Figure II-7.

Nationally, water use rates have been increasing over the last several decades and are expected to continue to do so. Certain industrial connections have been made or are about

FIGURE II - 7

GENERAL INCREASE IN AVERAGE WATER USE



Curve A - Affluent area with high transient and/or industrial demand.

Curve B - Average income area with nominal tourist and/or industrial demand.

After: Sweitzer and Flentje, BASIC WATER WORKS MANAGEMENT

(ARLINGTON: ACPPA, 1972)

to be made. For example, the GAF Binghamton facility will connect to the Binghamton-Johnson City STP very soon and discharge an average of 1 mgd of wastes (representing about 10 percent of the non-infiltration/inflow discharges to the plant). In Kirkwood, the new Frito Lay processing plant will discharge 0.29 mgd with up to 5,600 pounds of BOD. The volumetric flow from Frito Lay is equivalent to that from a population of 5,000 people and the BOD loading is equivalent to that from 30,000 people. The inflation adjusted price of water supply and sewage treatment to the consumer is expected to decline in the region (at secondary treatment levels) with an attendant increase in demand for water. Lastly, domestic water consumption is expected to increase as a result of a continued trend towards the use of appliances, such as washing machines, with high water use requirements.

The increase is not projected to continue indefinitely, however. Water use requirements should gradually level off as the demand for and total cost of purchase of water becomes large. This leveling of increases is a departure from the initial estimate, which projected continuous 10 percent per decade increases for 50 years.

There is no uniform agreement on the increase in projected water consumption rates. Some studies have indicated higher degrees of growth while others have projected no growth at all. A sample of such studies is summarized in Table II-7.

DERIVATION OF WASTEWATER FLOW PROJECTIONS

The flow projections presented in this section constituted the expected flows for all plans not employing nonstructural flow reduction measures. The assumptions used for each service area are given below along with the estimated 1973 per capita flows. These flow rates were based on STP records where possible or on estimates of typical Study Area flows based on water supply records (as previously discussed).

TABLE II-7

COMPARISON OF PER CAPITA WATER USE TRENDS AND PROJECTIONS

Study	Area	Per Capita Increases Reported	Equivalent 50-Year Increase to Year 2020
1. Linaweaver	USA	About 9%/decade increase since 1900 estimated.	54% (To 1960)
2. TSC	Southeastern New York State	0.5 gpcd per year increase projected equivalent to initial increase of 4½% per decade declining to 4% per decade.	22%
3. Holzmacher	Long Island, NY	General average increase of 2% per decade projected.	9%
4. Greeley & Hanson	Long Island, NY	6% per decade increases declining to 1½% per decade increases projected	15%
5. New Jersey	Northeastern New Jersey	9% per decade increases declining to 6% per decade increases projected	44%
6. BWWMS--Initial	Binghamton, NY	10% per decade projected	61%
7. BWWMS--Final	Binghamton, NY	10% per decade projected declining to 0% by 2000.	27%

REFERENCES

1. F. P. Linaweaver, et. al. A Study of Residential Water Use. (US HUD 1957).
2. Temporary State Commission on the Water Supply Needs of Southeastern New York. Scope of Public Water Supply Needs (1972).
3. Holzmacher, McLendon and Murrell. Suffolk County Comprehensive Water Supply Study.(1968).
4. Greeley & Hanson. Nassau County Comprehensive Water Supply Study. (1971).
5. New Jersey Division of Water Policy and Supply. Water Resource Circular 21. (1969).
6. US Army Corps of Engineers. Binghamton Wastewater Management Study. 1976
7. Ibid.

Binghamton-Johnson City Service Area

The current per capita flow rate of 115 gpcd was used, except for the following submunicipal areas:

1. V3 has mixed land use with a high degree of industrial activity (135 gpcd).

2. Areas C2 and K4 are residential (60 gpcd).

3. Areas K4 and C1 are expected to exhibit mixed land use (135 gpcd) by year 2000, with an intermediate rate of 98 gpcd in 1980.

A constant 7.5 mgd (Vernon O. Shumaker, 1974) is added to the projections as total I/I flows, based on the difference between estimated dry weather flows and the total flow recorded at the STP.

Endicott Service Area

The current per capita flow of 78 gpcd was used except for areas E1, U1, and U2. For area V3, 135 gpcd was used to reflect the high degree of industrial activity, and for areas V2, M1, M2, and M4, 60 gpcd was useed to reflect the predominantly residential character os these areas. A constant 0.7 mgd was added to the projections for infiltration/inflow.

Owego Village Service Area

The current per capita flow rate of 71 gpcd was used with 0.5 mgd of wet weather infiltration.

East Owego Service Area

The projections assume that submunicipal areas OT2 and OT9 will shift from residential use (60 gpcd) to mixed uses (135 gpcd) by year 2000, in addition to experiencing the per decade, general flow rate increments. Area OT1 would remain residential.

West Owego Service Area

The waste flow projections were based on the following:

1. Constant 140,000 gpd from the IBM Owego plant.

2. 60 gpcd for residential users, with decade rate increases.

3. Existing commercial flows of 70,000 gpd, with increases in proportion to total town population growth.

4. About 100 acres of land convenient for industral development (1200 gpd/acre, with decade increases) developed uniformly to year 2020 and would be sewered after year 1990.

These assumptions, instituted during Stage III-1, resulted in a major change from previous iterations.

Infiltration to Owego Town No. 1 plant was expected to become negligible by 1977. Average yearly treatment plant flows at West Owego have been steadily declining over the past few years. For example, the average flows were 0.416 mgd, 0.384 mgd, and 0.21 mgd in 1972, 1973, and 1974, respectively. This decline has been attributed to control of infiltration and inflow. This I/I reduction was achieved through a yearly sewer maintenance budget, which was instituted a few years ago. The assumption of the wasteflow projection is that this maintenance will continue to 1977 by which time only insignificant infiltration will be experienced.

Chenango Valley Service Area

The final flow projections were derived in Stage III-2 and resulted from the changes in the 100 percent sewering assumption investigations. During previous iterations, the following was assumed: (1) 100 percent sewering of the Valley was projected; (2) Dickinson was placed in the Binghamton-Johnson City service area; and (3) Areas CH2 and F2 were assumed to remain residential (60 gpcd), and areas CH3 and F1 were anticipated to have land usage intermediate between completely mixed and residential extremes (average of 135 and 60 gpcd, or 98 gpcd).

The modification of the 100 percent sewering assumption in Stage III-1 was assumed to impact upon the earlier projections by withdrawing land from sewering with completely residential development. Thus, the adjustment was made by subtracting the population not sewered, at 60 gpcd (adjusted for the decade rate increases), from the previous flow projections.

Additionally, a small contribution of 45,000 gpd was added from Dickinson.

All the above per capita rates were increased 10 percent, 10 percent, and 5 percent, for 1980, 1990, and 2000, respectively, to reflect the general increase in projected water consumption as previously discussed. No increase in per capita flows was projected after the year 2000. The expected wastewater flow projections assuming no flow reduction measures are presented in Table II-8.

LOW FLOW PROJECTIONS DUE TO NONSTRUCTURAL MEASURES

Nonstructural methods for reduction in wastewater flow, and their effectiveness were also investigated. Ths implementation of nonstructural measures can be expected to reduce sewage discharges 20 to 30 percent by 2020, i.e., the measures would negate the projected per capita decade rate increases (10%-10%-5%) by maintaining per capita discharges at existing levels. The measures were found to be approximately 100 percent cost-effective for AWT systems although some small degree of cost-effectiveness could be expected for the other treatment alternatives.

The investigation into the potential effectiveness of various nonstructural measures considered pricing and public education, and, to a lesser degree, zoning and building codes, and industrial surcharges. The conclusion of the analysis was that a program of pricing, via metered water consumption, to recover the cost of sewage treatment from dischargers, supplemented with a low cost, public education program, could be quite effective in reducing flow to a treatment plant. The measures would only be fully effective in the AWT alternative because only that alternative would result in sewer rates high enough to compel users to lessen their water demands. Other researchers have found industrial surcharges on other parameters (e.g., BOD or SS) to be effective in reducing discharges, however, this study could not evaluate such a measure since the region is just beginning to compile an industrial inventory in compliance with their NPDES discharge permits.

Commonly found literature values for price elasticities of demand were relied upon as the basis of the pricing analysis. Elasticities of -0.25 and -0.6 were used for domestic and non-domestic users, respectively, for water demand.

TABLE II-8

WASTEWATER FLOW PROJECTIONS (mgd)

	1973	1980				2000				2020			
Service Area	Total[1]	RESidential	COMmercial	INDustrial	Total[1]	RES	COM	IND	Total[1]	RES	COM	IND	Total[1]
Binghamton-Johnson City[2]	18.3	7.1	2.4	3.8	20.8	8.4	2.7	4.5	23.2	8.7	2.8	4.8	23.9
Five Mile Point[5]	---	0.6	0.1	0.1	0.8	0.7	0.2	0.3	1.3	0.8	0.3	0.4	1.5
TOTAL Service Area	---	7.7	2.5	3.9	21.6	9.1	3.0	4.9	24.5	9.5	3.1	5.3	25.4
Endicott[3]	4.2	2.7	0.6	0.4	4.4	3.2	0.7	0.5	5.1	3.2	0.7	0.5	5.1
Vestal[6]	1.0	1.1	0.4	0.8	2.3	1.7	0.5	1.1	3.3	1.7	0.6	1.3	3.5
Nanticoke Valley[6]	---	0.4	---	---	0.4	0.5	---	---	0.5	0.6	---	---	0.6
TOTAL Service Area	---	4.2	1.0	1.2	7.1	5.4	1.2	1.6	8.9	5.5	1.3	1.7	9.2
Chenango Valley	---	1.1	0.3	0.1	1.5	1.4	0.5	0.1	2.0	1.6	0.5	0.1	2.2
East Owego	0.4	0.9	0.3	---	1.2	1.4	0.5	0.6	2.5	1.6	0.5	0.6	2.8
West Owego	0.2	0.05	0.002	0.15	0.2	0.3	0.02	0.2	0.5	0.3	0.1	0.3	0.7
Owego Village[4]	0.9	0.3	0.1	---	0.9	0.3	0.1	---	0.9	0.4	0.1	---	1.0
TOTAL	25.0	14.2	4.3	5.3	32.5	18.0	5.2	7.4	39.3	19.0	5.6	8.0	41.3

1. Total includes infiltration
2. Average infiltration of 7.5 mgd.
3. Average infiltration of 0.7 mgd.
4. Average infiltration of 0.5 mgd
5. Included in Binghamton-Johnson City Service Area by 1977.
6. Included in Endicott Service Area by 1977.

The analysis was executed during the early part of Stage III-1, when population and wasteflow projections from Stage II-2 (i.e., the projections not adjusted for the 100 percent sewering assumption and resultant engineering cost estimates) were used to estimate future flow rates. The analysis projected a series of price-demand curves for water based on a number of assumptions such as future personal income levels, and the inflation adjusted price of water supply. With a projected demand curve for water and sewer for the year 2000, the incremental prices (sewer rents) resulting from implementation of the various alternatives were used to estimate movement along the demand curve. The incremental price ($/1000 gallons) is that charge in excess of charges which would be levied for secondary treatment. Thus, as Table II-9 demonstrates, demand will not be greatly reduced by pricing with the secondary treatment alternatives. If secondary treatment were to be financed via any other mechanism than direct pricing, one would expect even higher per capita flow rates than projected as the "high flows." Thus, the analysis assumed that areas such as Endicott and Owego Town would convert to metered rates for sewer rents.

Education programs which demonstrate to the residential users the various savings available to his water, sewer and water heating bills by installing "water-saving devices" could be effective in reducing demand. Faucet aerators, flow limiting shower heads, and low water using toilets or plastic bottles in toilet storage tanks are examples of these devices. The devices were assumed to be cost-effective to the residential user if he could recover his capital expenditure for device installation in five to six years. Whereas pricing results in movement along the projected demand curves, education results in actual shifts in demand.

Education programs to convince residential users to save more water than that which they find personally cost-effective have been found to have only short-term effects. Education programs for non-residential users are not effective because such users respond only to the price of water. With 100 percent effectiveness of nonstructural measures (no increase in per capita flows throughout the planning period), the projected wastewater flows would be those depicted in Table II-10.

TABLE II-9

EFFECTIVENESS OF NONSTRUCTURAL MEASURES IN YEAR 2000

	ΔQ_p (MGD)*	ΔQ_1 (MGD)*	Percent Achievement
Binghamton			
2d Treatment	0.00	1.10	23
5 mg/l Plan	0.22	1.10	31
BIO AWT	1.61	1.26	62
P/C AWT	2.28	1.26	75
Endicott			
2d Treatment	0.00	0.70	37
BIO AWT	0.86	0.78	86
P/C AWT	1.35	0.78	100
Chenango Valley			
2d Treatment	0.00	0.21	46
5 mg/l Plan	0.03	0.21	53
BIO AWT	0.27	0.22	100
P/C AWT	0.36	0.22	100
East Owego			
2d Treatment	0.00	0.18	35
BIO AWT	0.37	0.19	100
P/C AWT	0.42	0.19	100
West Owego			
2d Treatment	0.00	0.08	33
BIO AWT	0.15	0.09	100
P/C AWT	0.21	0.09	100
Owego Village			
2d Treatment	0.00	0.04	21
BIO AWT	0.14	0.05	100
P/C AWT	0.20	0.05	100

ΔQp -- Change in demand from pricing.

$\Delta Q1$ -- Change in demand from education program to encourage homeowners to install cost-effective plumbing devices to save water.

Percent Achievement -- Degree to which $\Delta Qp + \Delta Q1$ achieve the nonstructural objective of no increase in per capita flow rates.

* Based on Stage II-2 Projections.

TABLE II-10

STAGE III LOW FLOW PROJECTIONS WITH NONSTRUCTURAL MEASURES[1] (mgd)

Service Area	1973 Flow	1980 Amount Reduced	1980 Flow	2000 Amount Reduced	2000 Flow	2020 Amount Reduced	2020 Flow
Binghamton-Johnson City[2]	18.3	1.2	19.6	3.4	19.8	3.7	20.2
Five Mile Point[3]	----	0.1	0.7	0.3	1.0	0.3	1.2
TOTAL Service Area	----	1.3	20.3	3.7	20.8	4.0	21.4
Endicott[4]	4.2	0.3	4.1	0.9	4.2	0.9	4.2
Vestal[5]	1.0	0.2	2.1	0.7	2.6	0.8	2.7
Nanticoke Valley[5]	----	0.0	0.4	0.1	0.4	0.2	0.4
TOTAL Service Area	----	0.5	6.6	1.7	7.2	1.9	7.3
Chenango Valley	----	0.1	1.4	0.4	1.6	0.4	1.8
East Owego	0.4	0.1	1.1	0.5	2.0	0.6	2.2
West Owego	0.2	0.0	0.2	0.1	0.4	0.1	0.6
Owego Village[6]	0.9	0.0	0.9	0.0	0.9	0.1	0.9
TOTAL URBAN STUDY AREA	25.0	2.0	30.5	6.4	32.9	7.1	34.2

1. No increase in current per capita flow.
2. Includes 7.5 mgd average daily infiltration.
3. Included in Binghamton-Johnson City service area by 1977.
4. Includes 0.7 mgd average daily infiltration.
5. Included in Endicott service area by 1977.
6. Includes 0.5 mgd wet weather infiltration.

COST AND ECONOMIC ANALYSIS ASSUMPTIONS AND METHODOLOGY

This section presents a discussion of the assumptions and methodologies employed in the economic analyses used throughout the Study. Up through Stage II-1 only preliminary cost estimates were made based on national averages. Detailed design and cost estimates were initiated in Stage II-2. In general, the basic methodologies established in Stage II-2 were not changed in the following iterations. Where refinements were made from one iteration to another, note of this is made in the following text.

INTEREST RATE

Interest rates were used in the following three analyses:

1. Present worth or cost effectiveness.
2. Phasing of construction.
3. Cash flow.

The intention of the present worth analysis was to allow a comparison of the cost of alternatives to the nation, and the use of interest rate enables a common base expression of both capital and operating and maintenance costs. Thus, as interest rate rises, the cost of capital expenditures is increased, favoring solutions which either delay capital expenditures into the future, or are characterized by low capital costs and higher operating and maintenance costs. This relationship is also pertinent to the phasing analysis.

For preliminary cost estimation (Stage I-3, Stage II-1), an interest rate of 6 percent was used. For the more detailed design and costs involved in Stages II-2 and II-1, the rate used was 5 7/8 percent as mandated by the Water Resources Council. The prescribed rate was revised by the WRC to 6 1/8 percent in 1975 and all final plans (Stage III-2) reflect this rate.

Interest rates were also used in the cash flow analysis, which estimated the actual local expenditures for each alternative, over the 50 year economic life (1977 to 2027). In this analysis, it was assumed that the capital costs would be covered by EPA and NYSDEC grants at the maximum allowable rates (75 percent and 12.5 percent, respectively), and that the remainder would be funded by local agencies' bonds.

PRELIMINARY COST ESTIMATION

Methodology

Costs for wastewater management strategies were first developed in Stage I-3. For this and the following iteration, Stage II-1, preliminary costs were developed as described below. Only general Study Area characteristics were taken into account such as average STP flows.

Costs for treatment facilities and interceptors were compiled from a variety of sources which summarized national averages for force mains, pump stations, and various treatment processes. The majority of the estimates were based on EPA publications and updated to December 1974 using a projected estimate of the ENR Construction Index. Estimates were made for each treatment plant, providing additional capacities and treatment processes as necessary. Savings in O&M costs from closing of existing facilities were also estimated from national averages. Each interceptor was estimated separately, using the most probable route from the wastewater management area to the appropriate treatment plant. All capital costs were increased by 30 percent to reflect the costs of engineering fees and contingencies.

For land treatment systems, the on-site capital and O&M costs were based on "Wastewater Treatment and Reuse by Land Application" (Pound and Crites, EPA, 1973). Two application modes were considered: spray irrigation and overland runoff.

The application rate of 1.7 inches per week for 26 weeks was based on the nitrogen uptake of vegetation growing on the disposal site (200 lb/acre for forest or grass). Due to the rather severe winters, the application period was restricted to 26 weeks (April through September). Cost for land was assumed to be $500 per acre which is typical for the Bicounty Area.

All capital and O&M costs were incremental over current costs associated with existing treatment plants. That is, they represented the additional costs associated with the proposed schemes. The design economic life for treatment plants and interceptors was 25 years and 50 years, respectively, at 6 percent interest. No salvage value was assumed. Local costs were assumed to be 12.5 percent of capital and 100 percent O&M.

Costs for each scheme included sludge handling costs, also based on national averages. For all plants below 30 mgd, anaerobic digestion followed by dewatering on drying beds and trucking to landfill was assumed. For larger plants, vacuum filtration and incineration were assumed to follow anaerobic digestion.

Cost Estimating Data

For the preliminary schemes developed in Stage I-3 and Stage II-1, cost data was assembled from a number of sources as indicated previously. The derived cost equations used for these preliminary analyses are summarized below. Estimated cost indices for December 1974 were the Engineering News Record Construction Cost Index of 2135 and the Consumer Price Index of 161. Cost data were generally reported in the literature as cost curves, and the equations given below were developed from these curves. Capital cost equations did not include engineering and design fees and contingencies.

Secondary Biological Treatment.

The secondary treatment process included preliminary treatment, primary sedimentation, secondary aeration, sludge handling, and chlorination. Smith (1972) has developed cost curves for estimating capital costs and Eilers (1970) has developed cost curves for estimating operating and maintenance costs. The capital cost included the cost for the above mentioned unit operations and processes, and costs for necessary appurtenances (piping, pumps, electrical connections, etc.). The cost equations for estimation of capital cost and O&M cost, updated to December 1974 dollars, follow.

Capital Cost:

$$C1 = 1.351\ Q^{0.485} \quad (1.0 \text{ to } 2.0 \text{ mgd})$$

$$C1 = 1.239\ Q^{0.611} \quad (2.0 \text{ to } 5.0 \text{ mgd})$$

$$C1 = 0.981\ Q^{0.748} \quad (5.0 \text{ to } 30.0 \text{ mgd})$$

$$C2 = 1.53\ Q^{0.699} \quad (30.0 \text{ to } 100.0 \text{ mgd})$$

Q is an average flow in mgd. C1 and C2 represent capital cost in million dollars for sludge handling schemes 1 and 2, respectively. In sludge handling scheme 1, anaerobically diegested sludge is dewatered over sludge drying beds and then trucked away for disposal. In scheme 2, anaerobically digested sludge is vacuum filtered and incinerated and then the ash is trucked away for final disposal. This capital cost does not include the cost for final disposal.

Operations and maintenance costs (dollars per year) were calculated using the following equation:

$$O\&M = 49407\ Q^{0.776} \quad \text{dollars per year}$$

O&M cost was updated using Consumer Price Index 151.

The following equations were developed from cost curves for estimating capital cost for upgrading primary to secondary treatment:

$$C = 0.848\ Q^{0.513} \quad (0.5 \text{ to } 2.0 \text{ mgd})$$

$$C = 0.741\ Q^{0.679} \quad (2.0 \text{ to } 5.0 \text{ mgd})$$

$$C = 0.591\ Q^{0.832} \quad (5.0 \text{ to } 10.0 \text{ mgd})$$

$$C = 0.383\ Q^{1.02} \quad (10.0 \text{ to } 40.0 \text{ mgd})$$

C represents the capital cost in million dollars and Q is an average flow, mgd.

Biological Based AWT Processes.

Smith (1970, 1972), Smith and Eilers (1970), and Yang (1974) have developed cost equations and curves for calculating capital and O&M costs for tertiary treatment processes. The cost equations for these tertiary treatment processes are presented below. C represents the capital cost in

million dollars, O&M represents the operation and maintenance costs in dollars per year, and Q is the average flow in mgd.

1. Multimedia Filtration -- Flow: 1 to 100 mgd

$$C = 0.161\ Q^{0.66}$$

$$O\&M = 22951\ Q^{0.62}$$

2. Nitrification:

$$C = 0.360\ Q^{0.485} \quad (1.0 \text{ to } 2.0 \text{ mgd})$$

$$C = 0.320\ Q^{0.66} \quad (2.0 \text{ to } 5.0 \text{ mgd})$$

$$C = 0.267\ Q^{0.772} \quad (5.0 \text{ to } 10.0 \text{ mgd})$$

$$C = 0.234\ Q^{0.829} \quad (10.0 \text{ to } 30.0 \text{ mgd})$$

$$C = 0.205\ Q^{0.863} \quad (30.0 \text{ to } 100.0 \text{ mgd})$$

$$O\&M = 31834\ Q^{0.385} \quad (1.0 \text{ to } 2.0 \text{ mgd})$$

$$O\&M = 25214\ Q^{0.626} \quad (2.0 \text{ to } 5.0 \text{ mgd})$$

$$O\&M = 23489\ Q^{0.738} \quad (5.0 \text{ to } 10.0 \text{ mgd})$$

$$O\&M = 17516\ Q^{0.855} \quad (10.0 \text{ to } 100.0 \text{ mgd})$$

3. Denitrification:

$$C = 0.338\ Q^{0.397} \quad (1.0 \text{ to } 2.0 \text{ mgd})$$

$$C = 0.302\ Q^{0.560} \quad (2.0 \text{ to } 5.0 \text{ mgd})$$

$$C = 0.254\ Q^{0.667} \quad (5.0 \text{ to } 10.0 \text{ mgd})$$

$$C = 0.191\ Q^{0.790} \quad (10.03 \text{ to } 30.0 \text{ mgd})$$

$$C = 0.142\ Q^{0.879} \quad (30.0 \text{ to } 100.0 \text{ mgd})$$

$$O\&M = 37645\ Q^{0.439} \quad (1.0 \text{ to } 2.0 \text{ mgd})$$

$$O\&M = 32491\ Q^{0.652} \quad (2.0 \text{ to } 5.0 \text{ mgd})$$

$$O\&M = 26280\ Q^{0.784} \quad (5.0 \text{ to } 10.0 \text{ mgd})$$

$$O\&M = 20223\ Q^{0.898} \quad (10.0 \text{ to } 100.0 \text{ mgd})$$

4. Chemical Addition for Phosphorus Removal:

$$C = 0.230\ Q^{0.74} \quad (1.0 \text{ to } 10.0 \text{ mgd})$$

$$C = 0.1725\ Q^{0.825} \quad (10.0 \text{ to } 100.0 \text{ mgd})$$

$$O\&M = 35790\ Q^{0.55} \quad (1.0 \text{ to } 10.0 \text{ mgd})$$

$$O\&M = 22838\ Q^{0.775} \quad (10.0 \text{ to } 100.0 \text{ mgd})$$

5. Carbon Adsorption:

$$C = 0.641\ Q^{0.6} \quad (1.0 \text{ to } 10.0 \text{ mgd})$$

$$C = 0.378\ Q^{0.86} \quad (10.0 \text{ to } 100.0 \text{ mgd})$$

$$O\&M = 134072\ Q^{0.45} \quad (1.0 \text{ to } 10.0 \text{ mgd})$$

$$O\&M = 56810\ Q^{0.80} \quad (10.0 \text{ to } 100.0 \text{ mgd})$$

Physical/Chemcial Treatment.

The above equations were used for the filtration, chemical addition, and carbon adsorption processes. Costs for secondary P/C were based on Howe (1971).

Land Treatment.

Costs were prepared for two different application modes: a 1.7 inches per week application rate for six months with six months storage, and a 2 inches per week application rate for three months with only a one-week storage facility. The objective of the first method was to treat all secondary effluent by land application. Six months was used as the maximum application period because of the short frost-free season (140 days), poor drainage, and hilly terrain. The application rate was based on the allowable nitrogen loading of the soil. However, because of the prohibitively high cost of providing six months of winter storage, a second method was also studied. This latter alternative would employ land treatment only during the critical summer months (June, July, August), with discharge of secondary effluent to the streams during the remaining months. Using this approach only a one-week storage facility would be needed (for wet weather conditions, system repairs, etc).

On-site wastewater distribution costs were based on "Manual of Practice--Regulation of Sewers" (1973), and storage lagoons costs were obtained from W. W. Eckenfelder (1970). Transmission costs were determined using the interceptor cost equations presented in the following section.

For six months application (year round pumping) the supplemental cost in dollars is: 37,020 Q (Q in mgd). For three months application and pumping, the supplemental cost in dollars is: 9,255 Q (Q in mgd).

Transmission Mains (Interceptors).

Smith has developed equations for estimating the capital cost of the pipeline. The cost relation is as follows in terms of December 1974 dollars (ENR 2135).

Construction Cost, \$/mile = 2275 (ID + 2.04)1.38

Where ID = inside pipe diameter, inches.

Equations used for calculating the pipe diamaters were:

Gravity sewers: $ID = 23.01\ Q^{0.376}$ at slope = 0.001

Force main sewers: $ID = 13.665\ Q^{0.461}$

Where Q = average flow in mgd.

O&M = \$50 per year per mile.

This construction cost includes only normal excavation; cost for special steel sheeting, road crossing, rock excavation or any other special need costs were not included. Construction cost and O&M cost of pumping stations for pumping sewage are presented below:

Construction cost (\$) = $109240\ Q^{0.761}$

O&M (\$/yr) = $33625\ Q^{0.758}$

Operation and maintenance cost for pumping stations includes the power cost for a static head of 20 to 30 feet. For any additional head required in pumping, additional power cost would be needed at a supplemental power cost based on \$0.04 per horsepower-hour.

REFINED COST METHODOLOGY

Beginning in Stage II-2 of this Study, the cost estimates were performed independently of those which had been used in the previous iterations for the comparison of broad strategies. The earlier work depended solely on cost curves, which related costs to single independent variables, primarily that of wastewater flow, and were based neither on specific study area conditions nor preliminary designs of the works (interceptors, pump stations, force mains, treatment units, etc.) themselves. Virtually all of the cost estimates were made in Stage II-2. The only new estimates made in Stage III were those necessitated by the refinements which arose as the Study progressed. In making such refinements, however, the methodology did not change.

The major design and cost differences between Stage II-2 and III-2 were:

1. A measurement of the effectiveness of nonstructural measures was made.

2. Infiltration control costs were incorporated into the analysis.

3. Projected wastewater flows were reduced as previously described.

4. More accurate construction and O&M cost curves for the Chenango Valley STP were used (to allow refinement of costs for all STP's to be near the same degree of accuracy).

5. A separate cost for the outfall for the Chenango Valley STP was itemized.

6. Ten hours of storage for average flows (2.2 mgd) was provided at the Chenango pump station for plans regionalizing Chenango Valley with B-JC. The size of the force main was correspondingly reduced as a result of the equilization of normally peaked flows.

7. The estimated cost for inflow control for Owego Village was taken from the K. G. Woodward "Report on Combined Sewer System--Village of Owego" adjusted for ENR change. The control involved a slight diversion of some creeks which are influent to the sewer system during periods of rainfall.

8. Costs for microscreening treatment for the Village of Owego combined sewer overflows were scaled from the estimates for the City of Binghamton. Flows (up to 3 mgd) in excess of the hydraulic capacity of the Village STP would receive microscreening before discharge to the river.

9. The construction cost for the Owego Village to West Owego force main was slightly reduced.

The following changes were made between Stage III-1 and III-2 design and cost estimating.

1. Costs for a new Chenango Valley plant and later expansions were itemized. The Chenango STP outfall, altered to discharge above the Route I-88 River Park, was estimated to cost $115,000 due to the special construction activities needed for that outfall (involving a highway and road crossing).

2. The Chenango force main costs were increased $800,000 (for plans regionalizing Chenango Valley with B-JC), to provide 10 hours of storage for peak flows (3.3 mgd) at the pumping station.

The Engineering News Record Index was changed from ENR 2135, Stage III-1, to ENR 2248, Stage III-2. Additionally, the interest rate was changed from 5 7/8 percent to 6 1/8 percent.

The development of previous alternatives and plans is discussed in detail in the Plan Formulation Appendix.

Capital Costs

Refinements in capital cost estimating consisted of two parts:

1. Preliminary design and sizing of the capital items; and,

2. Consideration of site conditions in the estimating of costs.

The design of capital items includes consideration of expected flow and pollutant loadings, economic design life, useful life and construction scheduling, all of which are detailed in other parts of this Appendix.

The following discussion first treats three elements of the capital cost estimations which were common to all the

estimates--Cost Index, Engineering & Contingencies Allowance, and Land Acquisition--and then describes the elements of cost considered in each of the types of installations required to form an alternative, e.g., interceptors, pump stations, treatment units, etc.

Cost Index.

In order to provide a common basis for all capital costs, a cost index must be chosen to represent costs prevailing as of the time of reporting. Although the start of the planning period, and the year in which initial construction is assumed for each alternative is 1977, it was decided that the dollars used should be prevailing ones, to avoid the hazards of predicting changes in relative construction costs.

Consideration was given to both the EPA cost indices (Sewer and Sewage Treatment Plant) and the Engineering News Record (ENR) construction cost index. Examination of past trends in these indices shown that they are subject to relative variations year to year, but that on the whole the increases over an extended period (1967 to 1974) are about equal, with the EPA indices having increased slightly more than the ENR index as shown in Table II-11.

TABLE II-11

COMPARISON OF CONSTRUCTION COST INDICES

Index	Late 1967 Value	Late 1974 Value	Percent
EPA Sewer (US Average)	126	238	89
EPA STP (US Average)	121	224	85
ENR Construction (20 US Cities Average)	1170	2099	80

Since the ENR index is the one most commonly used, and since it had been used in Stages I and II work, it was decided to use this value in updating experienced costs.

The Stage II-1 work projected costs to December 1974 using a (then) projected ENR index of 2135 (i.e., slightly higher than the experienced value of 2099). Stages II-2 and III-1 continued to use ENR 2135. For final Stage III-2 planning, however (the latest published (July 1975) ENR Cost Index (20 US cities average) of 2248 was used.

Engineering and Contingencies.

All individual construction cost estimates attempted to predict actual contractors' prices for the given work. To such figures must be added the cost of engineering services, administrative charges, unforseen expenditures, etc., commonly referred to as "Engineering and Contingencies." With estimates prepared after a construction design is underway or completed, these costs can be identified individually and fairly precisely. However, with the preliminary nature of the present designs, these costs are estimated as a single item, which experience has shown to approximate 30 percent of construction cost. Thus, capital cost is 130 percent of estimated construction cost.

Land Acquisition.

Land acquisition is an item which is hypothetically needed for the construction of all capital items, i.e., pump stations, force mains, interceptors, treatment plants, and land disposal sites. However, these costs will be negligible, or not applicable, in most of the alternatives, because (1) treatment plant expansions are on property already owned by the STP operating agencies; and (2) interceptors and force mains are generally on public lands, such as highway rights-of-way, and even if not, land costs are only for easement purchase and are not great.

Thus, land acquisition costs were included only for new pump stations and a new treatment plant at Chenango Valley and for land treatment sites. Costs were based on an estimate of land required and the prevailing land costs near the site. Land costs were not included for sludge disposal by land application, since this alternative would use productive agricultural land, (except for the small amount of land required for storage).

Interceptors and Force Mains.

Required interceptors and force main routes were laid out with consideration given to both economics and, where applicable, the avoidance of unusual environmental damage or adverse impacts. Sizing of pipes was based on economic design life and flow peaking factors. Capital cost estimates were based on experienced basic costs for pipe laying, on a per lineal foot basis at expected trench depths, plus manholes, highway crossings, river crossings, and dewatering as separate cost items. Soil borings were not taken, and

costs assume average excavation conditions. If rock excavation is required, the costs for this work would increase substantially.

Pump Stations.

Pump station cost estimating was based on preliminary structure sizing, and accounted for excavation, piling (if necessary), dewatering, and sheeting. Pumps and control equipment were estimated as a single price based on preliminary pump sizing, plus piping as a separate item. In one case, expansion of the Binghamton-Johnson City raw waste pumping capacities, the costs are based on an estimate of pump impeller change only, as motors are sufficiently sized for future flows.

Secondary Treatment Expansion.

Secondary treatment expansions of existing primary or secondary plants were designed to be consistent with the type of system, i.e., trickling filter or activated sludge, already installed or planned for installation at each site. The question of the ability of trickling filters to meet secondary treatment requirements is discussed in Chapter V of this Appendix.

In making cost estimates, required major units were sized and experience costs applied to these units, such as concrete, blowers, trickling filter equipment, clarifier mechanisms, vacuum filters, and digesters. Costs for yard piping attempted to account for actual existing plant layout, but detailed analyses of yard piping costs were not made. Each estimate completed in the above manner was compared to overall cost curves for a rough check on completeness of the estimate.

New Secondary Treatment Plant.

In only one scheme, i.e., separate treatment for the presently unsewered Chenango Valley area, was a new treatment plant required. Costs and dimensions of each unit process were itemized. Previously developed cost curves were used and checked against the costs for treatment plant expansions done on a unit-by-unit basis.

Nitrification Units.

Tankage and equipment required for suspended growth nitrification is identical in type to that used in conventional activated sludge plants. For this reason, design included a preliminary sizing of aeration tanks and clarifiers, and aeration equipment, and costs were developed in the same manner as the activated sludge secondary treatment plant expansions.

Denitrification Units (for Biological Based AWT).

Suspended growth denitrification is similar to tankage and equipment design to nitrification, except the reactor itself is not aerated, but must be degasified (by aeration) prior to clarification. Each of the required units was sized, and cost estimates were performed in a similar fashion to the activated sludge secondary treatment plant expansions.

Phosphorus Removal (Biological and P/C AWT).

Phosphorus removal would be accomplished by alum addition prior to clarification. Clarifiers would be required for other process steps so the only capital cost would involve the alum addition equipment. Estimate of this minor cost was based on cost curves.

Rapid Filtration (Biological and P/C AWT).

Filtration represented a small cost in relation to the total AWT treatment system, so cost curves were used.

Activated Carbon (Biological and P/C AWT).

The sizes of activated carbon systems (contact and regeneration units), were based on dosage rates and regeneration times designed for each application (see Chapter V). Cost estimates were based on those reported in the literature for similarly sized systems, in terms of dosage rate, contact reactor sizes, and carbon regeneration rates. Costs of activated carbon processed for biological-based and P/C AWT differed greatly for the same hydraulic flow rate.

Breakpoint Chlorination (P/C AWT only).

The P/C AWT systems were designed for installation in 1985, as a conversion from biological secondary treatment. Since breakpoint chlorination requires only 30 minutes hydraulic detention time, it was assumed this could be provided by units which would be abandoned, i.e., aeration tanks or secondary clarifiers. The capital cost of this conversion would be negligible, and so was not included in the cost estimate.

Infiltration Control.

The costs of infiltration control for the City of Binghamton system were evaluated in Stage III-1. Because of the cost savings and environmental benefits, an economically efficient level of infiltration control was defined for each plan. The cost for control of infiltration in the City of Binghamton broken down into nine separate projects, was taken from an infiltration/inflow analysis study (Schumaker, 1974). A curve showing the cost of infiltration control versus the flow reduction achieved is presented in Figure II-8. This curve was used in the cost-effectiveness analysis to determine the degree of infiltration control economically achievable for each of the plans.

Combined Sewer Overflow Treatment.

The design of the four Alternatives for combined sewer overflow treatment in the Binghamton-Johnson City Service Area is discussed in detail in Chapter VII and included:

1. on-site microscreening, disinfection, and discharge.

2. on-site air flotation, disinfection, and discharge.

3. on-site storage, plus post-storm release to sewer system for treatment at B-JC treatment plant.

4. centralized treatment using a modified biological secondary system.

Cost curves, related to flow rate or volume, were developed by itemizing costs for structures, major equipment plus installation, site work, pumps, etc., for the range of sizes considered. Costs for each individual overflow site were taken from that curve, using the actual design rate of volume at each overflow site.

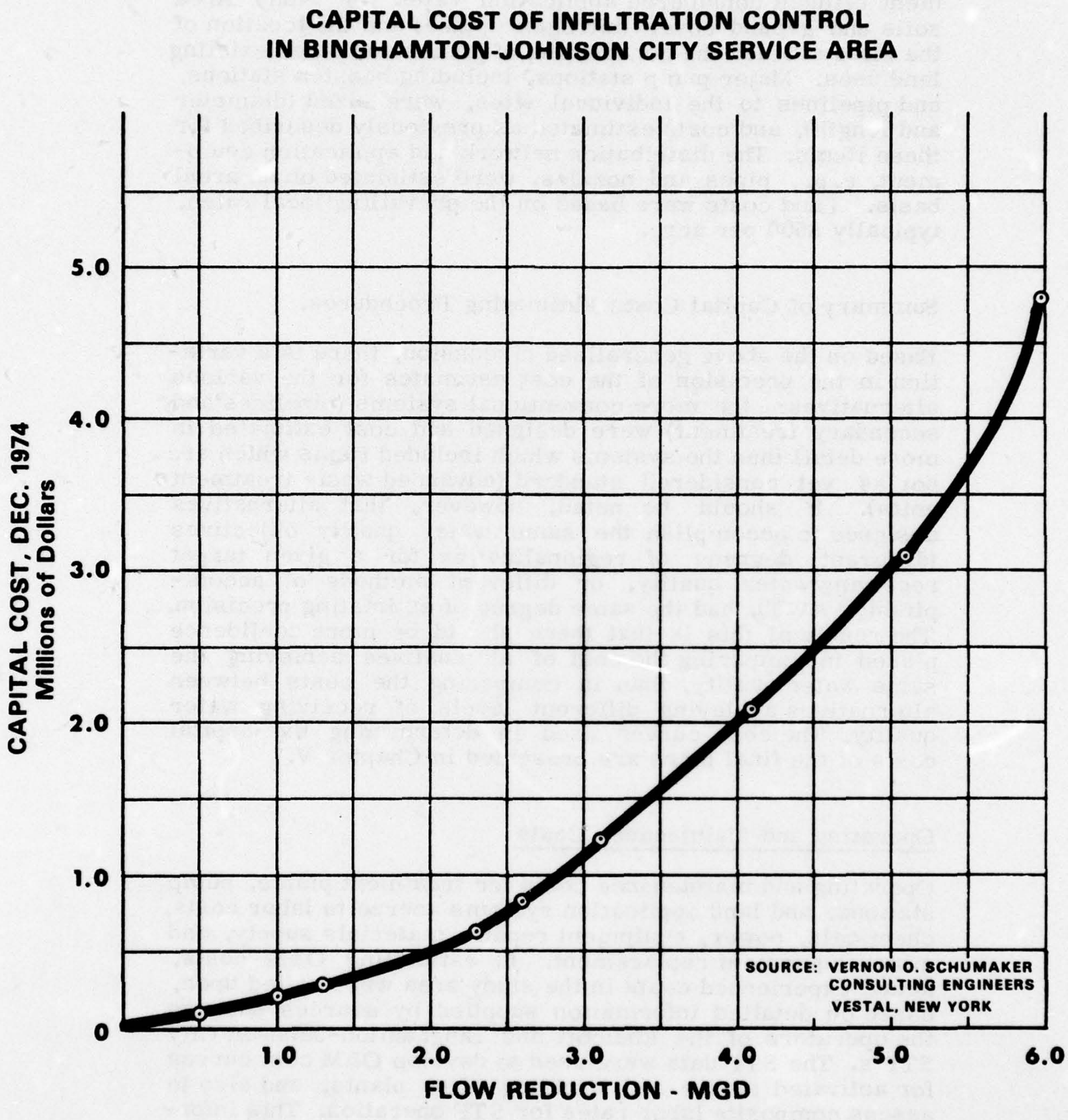

CAPITAL COST OF INFILTRATION CONTROL
IN BINGHAMTON-JOHNSON CITY SERVICE AREA
CAPITAL COST, DEC. 1974
Millions of Dollars
5.0
4.0
3.0
2.0
1.0
0
1.0
2.0
3.0
4.0
5.0
6.0
FLOW REDUCTION - MGD
SOURCE: VERNON O. SCHUMAKER
CONSULTING ENGINEERS
VESTAL, NEW YORK

FIGURE II-8

Land Treatment of Secondary Effluents.

The design of land treatment systems for secondary treatment effluent considered application rates for Study Area soils and ground cover (nitrogen uptake) and the location of the sites avoided the condemning of homes and other existing land uses. Major pump stations, including booster stations, and pipelines to the individual sites, were sized (diameter and length), and costs estimated as previously described for these items. The distribution network and application equipment, e.g., pipes and nozzles, were estimated on an areal basis. Land costs were based on the prevailing local rates, typically $500 per acre.

Summary of Capital Costs Estimating Procedures.

Based on the above generalized discussion, there is a variation in the precision of the cost estimates for the various alternatives: the more conventional systems (pipelines and secondary treatment) were designed and cost estimated in more detail than the systems which included items which are not as yet considered standard (advanced waste treatment units). It should be noted, however, that alternatives designed to accomplish the same water quality objectives (different degrees of regionalization for a given target receiving water quality, or different methods of accomplishing AWT), had the same degree of estimating precision. The result of this is that there should be more confidence placed in comparing the cost of alternatives achieving the same water quality, than in comparing the costs between alternatives achieving different levels of receiving water quality. The cost curves used in determining the capital costs of the final plans are presented in Chapter V.

Operating and Maintenance Costs

Operating and maintenance costs for treatment plants, pump stations, and land application systems accrue to labor costs, chemicals, power, equipment repair, materials supply, and minor equipment replacement. In estimating O&M costs, actual experienced costs in the study area were relied upon, based on detailed information supplied by sources such as the operators of the Endicott and Binghamton-Johnson City STP's. The STP data were used to develop O&M cost curves for activated sludge and trickling filter plants, and also to assess composite labor rates for STP operation. This information was supplemented by obtaining study area data on the remainder of the major costs involved in the operation of

treatment and disposal works, (including chemical costs for AST alternatives), power rates, and fuel costs.

The following discussion describes how this information was combined in the specific estimates for each major type of operating account.

Secondary Treatment.

During Stage II-2, an attempt was made to procure the 1974 O&M costs for all the existing secondary treatment plants, i.e., Binghamton-Johnson City (B-JC), Endicott, East Owego, and West Owego; however, only those from B-JC and Endicott were obtained. Total 1974 O&M costs for these STP's were:

B-JC	$685,000 (Budget, 1 Jan to 31 Dec 74)
Endicott	$291,553.71 (Actual, 1 Jun 73 to 31 May 74)

The B-JC costs were based on budget since 1974 was the first full year of that plant's operation at secondary treatment.

In using those costs to construct Study Area secondary treatment cost curves, cognizance was taken of the following:

1. Activated sludge plants, such as B-JC, are higher in O&M costs than trickling filter plants, such as Endicott, for a given size plant; and

2. O&M costs are strongly dependent on STP capacity, not to actual flow rates in an under-loaded plant. The reason for this is that a large portion of the O&M (greater than 50 percent for B-JC and Endicott) is for direct labor salaries and benefits, which will vary only slightly with hydraulic flow rate; in addition, several other major items, such as general building maintenance, equipment repair, general operating supplies, insurance, etc., will not vary significantly with flow rate. The only secondary treatment costs which will vary significantly with flow rate are utility costs, for pumping and aeration, and chemical costs for chlorination and sludge conditioning. These costs represent only about 20 percent of the total O&M, and since they are only partially dependent on actual flow rate, will not seriously affect the accuracy of assuming that O&M costs are proportional to the design capacity.

Since Study Area O&M costs were obtained from only one plant for each of the two types of secondary treatment, other sources had to be examined for the construction of cost

curves. Two such sources (Swetzer and Flentje, 1972, and Associated Engineering Services Ltd, et. al., 1973) indicated in the range of flows under study (1.0 to 30 mgd), that annual O&M costs could be related to capacity by an equation of the form:

$$\text{O\&M (\$)} = aQ^{b} \text{ ; where: } Q = \text{design capacity, mgd}$$
$$a = \text{constant}$$
$$b = \text{constant (economy of scale).}$$

Swetzer and Flentje (1972) evaluated "b" at 0.699, and information from the second reference above (Associated Engineering Ltd.) was used to compute a "b" of 0.76. The average of these numbers, 0.73, was used in this study. The resulting curves drawn through the 1974 B-JC and Endicott costs are shown on Figure II-9. The equations in Table II-12 summarize these curves.

TABLE II-12

COST EQUATIONS for SECONDARY TREATMENT O&M

Treatment	Annual O&M Q = Design Capacity (mgd) 1974 $
Activated Sludge	$\$ = 82,000\ Q^{0.73}$
Trickling Filter	$\$ = 67,000\ Q^{0.73}$

Labor Rates.

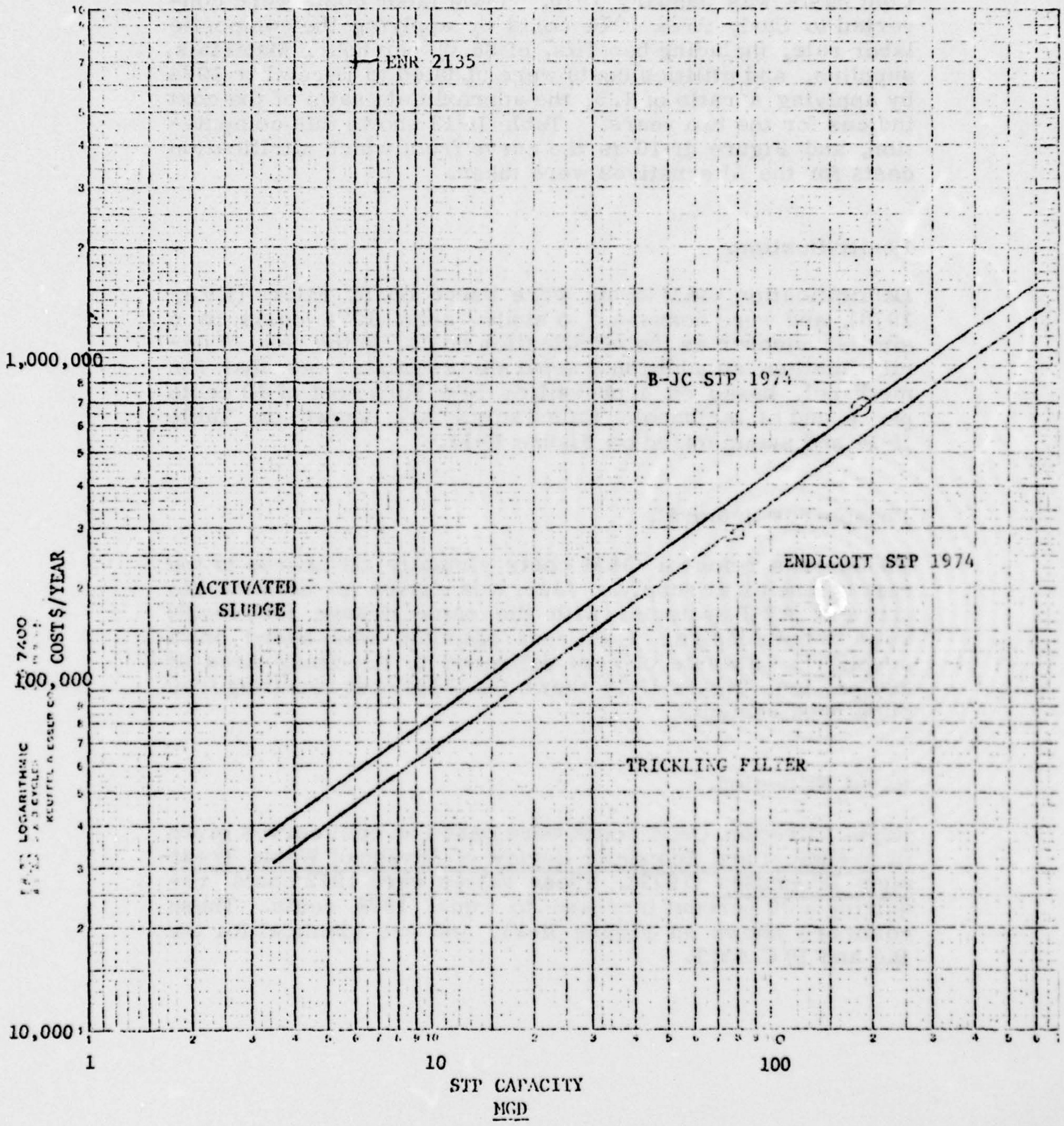
The composite labor rate at the Endicott STP, including salary plus benefits, was $6.20 per hour in 1974. Labor rates for STP operation were applied primarily in the estimates for nitrification, denitrification, and land application. A composite rate of $6.00 per hour was used.

Pump Stations.

Pump station O&M in any alternative was a small cost in relation to other O&M and capital costs. For this reason, cost curves were used to assign this cost, taken from R. Eilers (1973). Pump station O&M was computed to be (in December 1974 dollars): Cents/1000 Gallons = $2.38\ Q^{-0.263}$. This cost included power, labor, and maintenance items.

FIGURE II - 9

Operating and Maintenance Costs
SECONDARY TREATMENT



Nitrification.

Nitrification O&M cost estimates (based on R. Eilers, EPA, 1973), detailed the operation man-hours, maintenance man-hours, and a sum cost for materials, supplies and utilities. Cost basis was January 1970. O&M labor costs were converted to Study Area 1974 costs by applying the composite labor rate, including benefits, of $6.00 per hour. Materials, supplies, and utilities costs were updated to December 1974 by applying a ratio of 1.5, the approximate ratio of the cost indices for the two years. Table II-13 shows this computation, and Figure II-10 is the curve from which nitrification costs for the Alternatives were taken.

Denitrification.

Denitrification O&M costs were based on R. Eilers (EPA, 1973), and were converted to study area 1974 costs in a similar manner as the nitrification costs. In the case of denitrification, an additional separate estimate was that for methanol, based on a prevailing New York cost of 40 cents per pound of methanol. Calculations are shown on Table II-14 and summarized on Figure II-11.

Phosphorus Removal.

Phosphorus removal O&M costs virtually all accrue to the cost of chemical supply. Alum was chosen for both the Bio and P/C AWT systems, with the same dosage (200 mg/l) used in both designs. O&M cost estimate was based on a supplier's estimate of cost delivered to the Study Area at $80 per ton. Figure II-12 shows the O&M cost resulting from alum requirements.

Rapid Filtration.

Rapid filtration O&M costs were based on cost curves given in Technical and Economic Review of Advanced Waste Treatment Processes (1973). These values were 1972 costs, requiring a 30 percent increase to equal 1974 costs. These costs are shown on Figure II-13, and are identical for the Bio and P/C AWT.

TABLE II-13

NITRIFICATION O&M COSTS [1]
(1973)

FLOW, mgd	1	10	100
LABOR, Man-hours/year	2,675	6,320	29,300
@ $6.00/hour--$/year	16,100	38,000	175,000
MATERIALS, Supplies, Utilities--1970 $	10,110	28,220	86,300
MATERIALS, Supplies, Utilities--to 1974 (x 1.5)	15,000	42,200	129,000
Sub-Total	31,200	80,200	304,000
Total (Including Yard, Lab, Admin @ 13.2%)	35,200	91,000	344,000

1/ Eilers, 1973

FIGURE II - 10

NITRIFICATION O & M COSTS

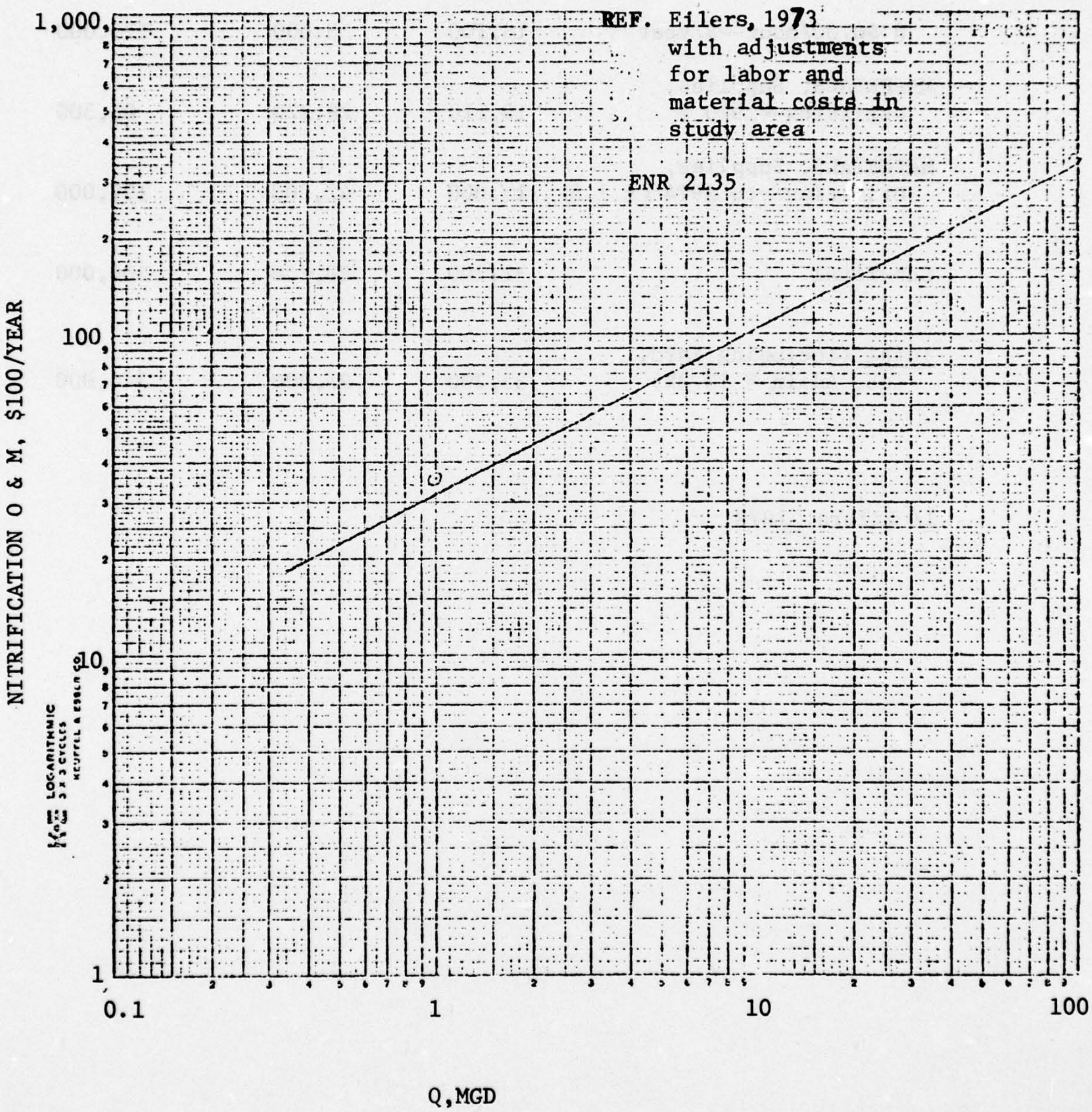


TABLE II-14

DENITRIFICATION O&M COSTS[1]
(1973)

Flow, MGD	1	10	100
Labor, Man-hours/year	3,041	6,247	26,900
@ $6.00/hour--$/year	18,246	37,400	161,400
Materials, Supplies	3,540	12,600	79,370
Materials, Supplies @ 1974 Cost (x 1.5)	5,310	18,900	119,000
Methanol Cost	11,080	110,800	1,108,000
Sub-Total	34,600	167,200	1,388,400
Total (Including 1.13 for Yard, Lab, Admin)	39,100	188,900	1,568,900

1/ Eilers, 1973

FIGURE II - 11

DENITRIFICATION O & M COSTS

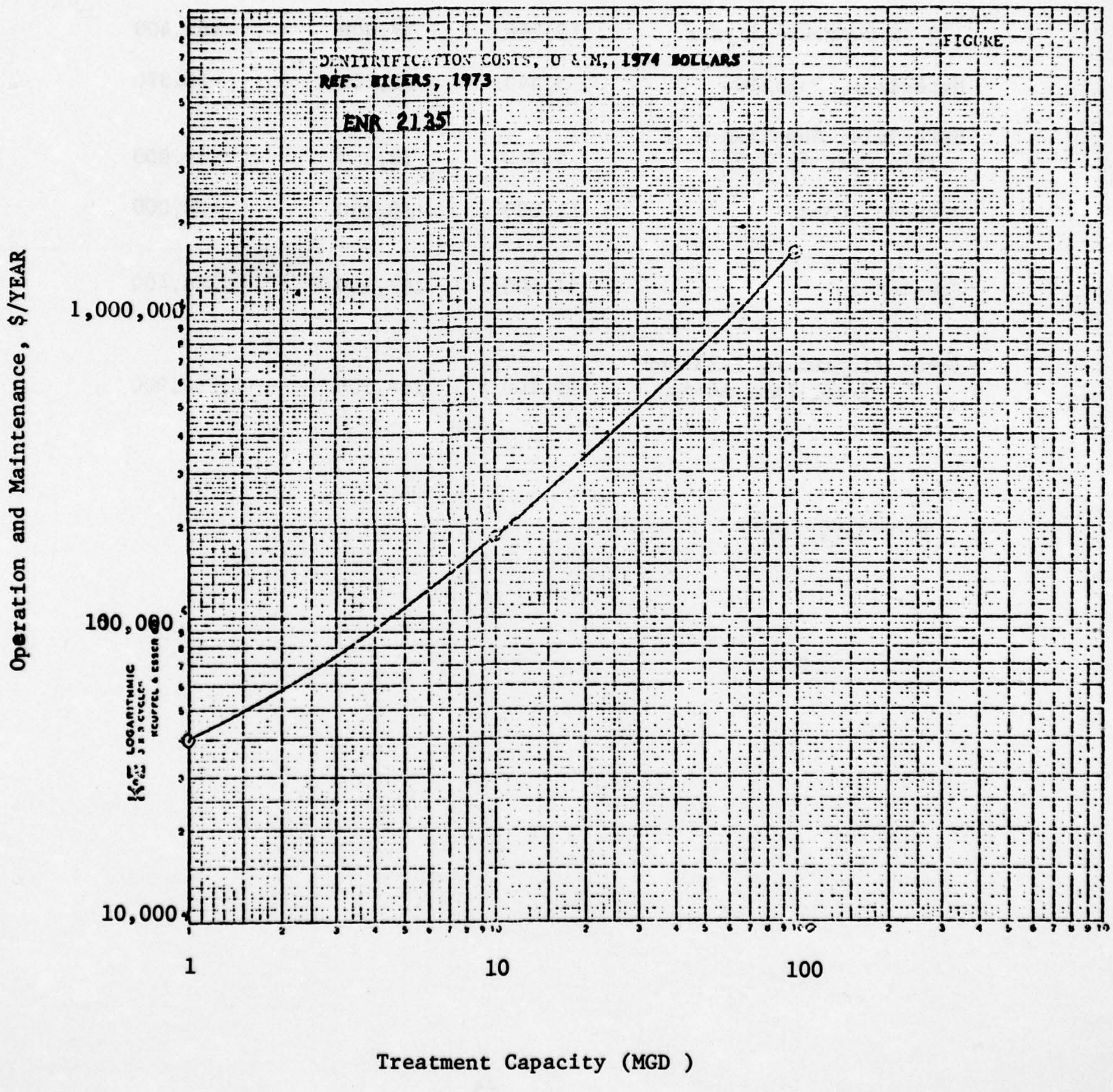
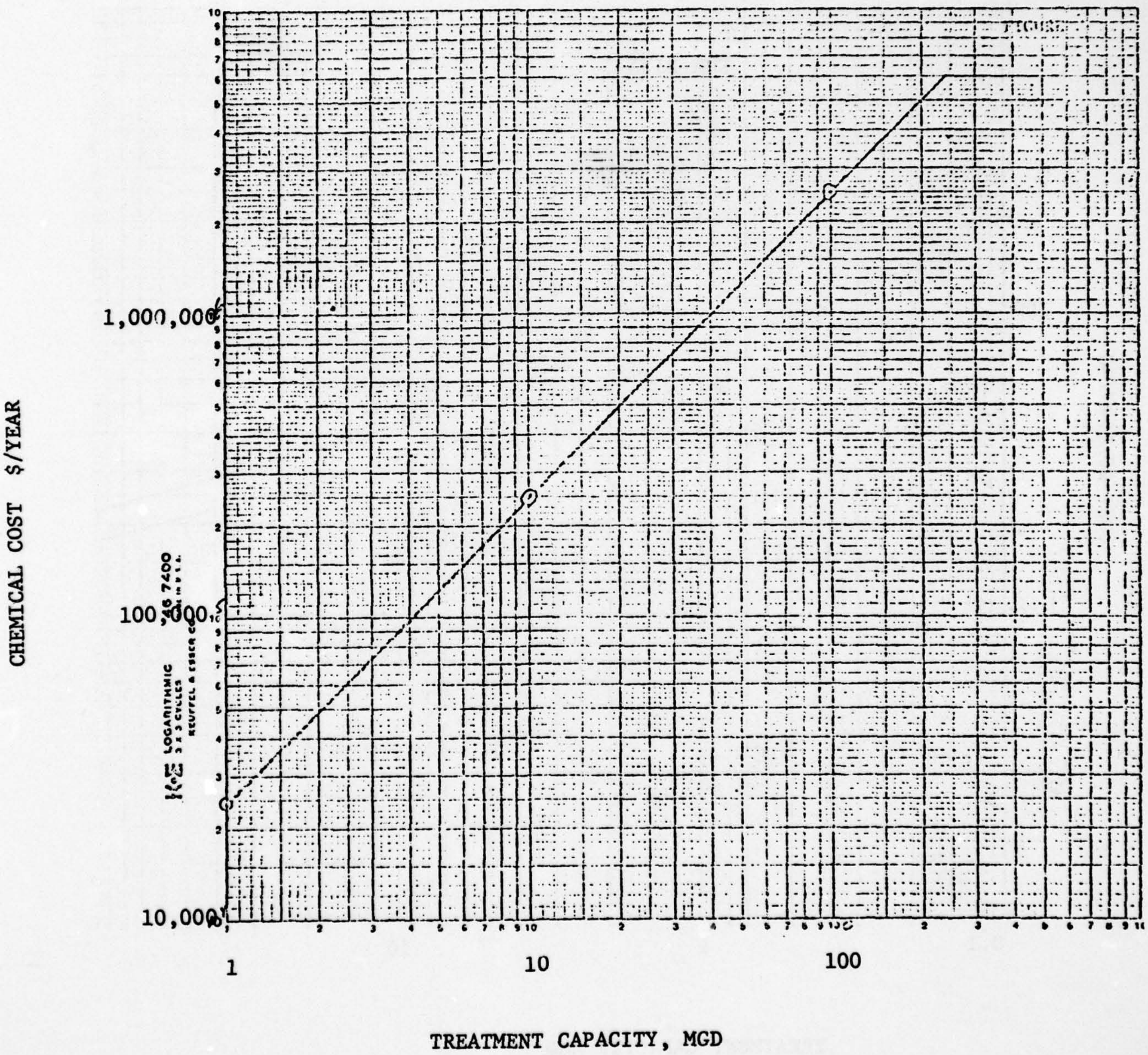


FIGURE II - 12

PHOSPHORUS REMOVAL O & M
Alum @ $80/ton)



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BINGHAMTON WASTEWATER MANAGEMENT STUDY. DESIGN AND COST APPENDI--ETC(U)
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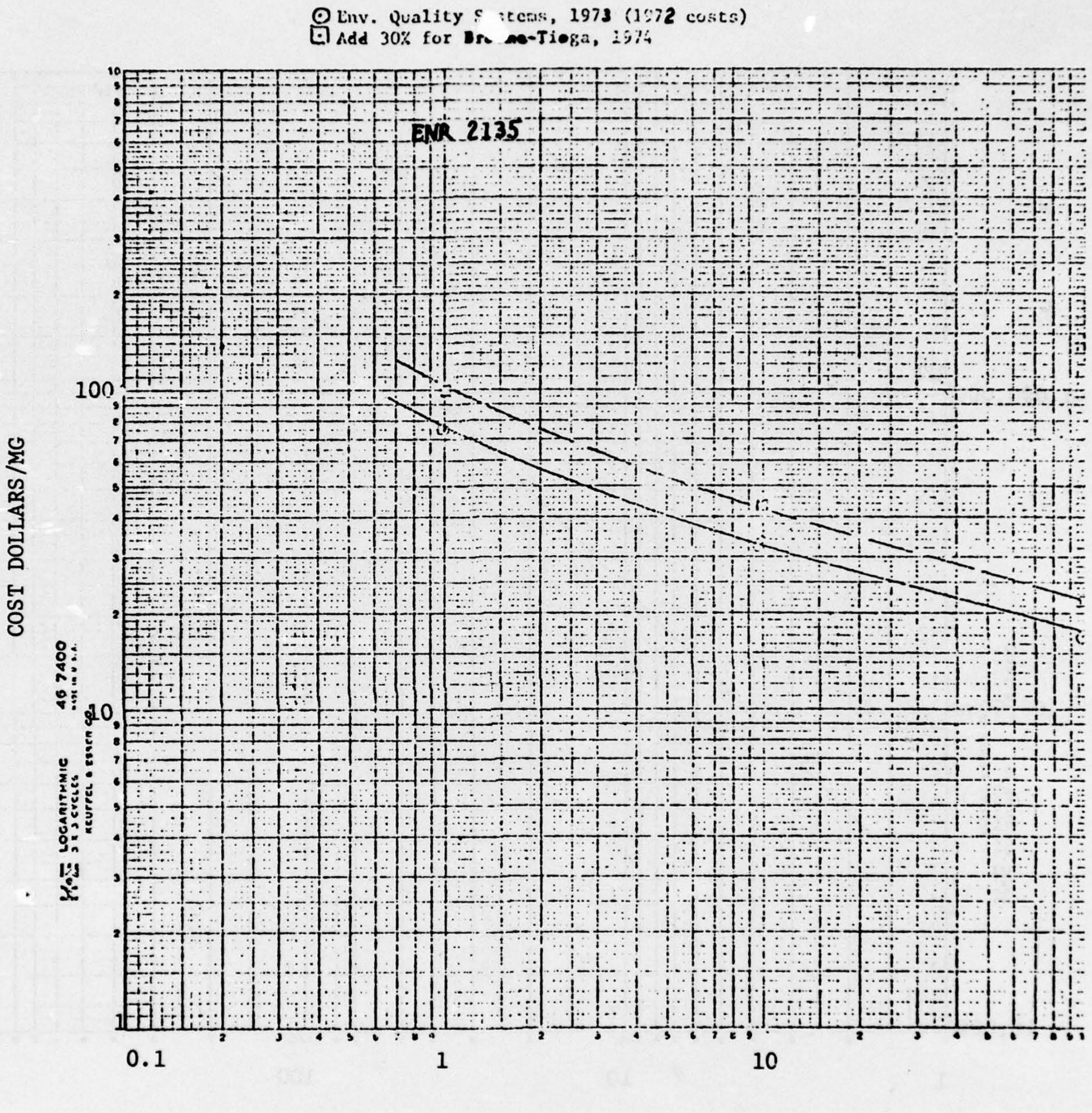
FIGURE II - 13

RAPID FILTRATION O & M



Activated Carbon.

Design of the activated carbon systems (Chapter V) differentiated between the Bio and P/C AWT, in that the P/C system required greater COD removals in the activated carbon step than the Bio system (since the Bio system accomplishes most of the required COD removal in the secondary and nitrification processes). Thus, the P/C system has larger treatment units, requiring more O&M labor, higher carbon attrition rates, and higher regeneration (fuel) costs.

Technical and Economic Review of Advanced Waste Treatment Processes was the basic document used to determine activated carbon O&M costs. That report assumed treatment prior to activated carbon virtually identical to that used in this Study for Bio AWT, and a carbon dosage and contact time identical (250 lb/MG and 15 minutes, respectively) to those estimated to be necessary for this Study. For this reason, the Bio AWT O&M was taken directly from the above reference with an adjustmenent for cost index changes (30 percent).

No published cost curve information could be found for the P/C AWT activated carbon system designed for this study, i.e., 50 minute contact time and dosage of 1800 lb/MG. For this reason, the detailed information given in the above reference for highly treated influent was adjusted to account for the design variables. The costs for carbon replacement and regeneration fuel are directly proportional to dosage rate, whereas other O&M costs (labor and materials) would realize an economy of scale *with system size*. P/C activated carbon O&M costs were thus computed as follows:

Fixed Costs:

1. Carbon, at 1800 lb/MG dosage

   7.5% attrition and $0.40 per lb = $54.00/mg

2. Regeneration Fuel at 5,000 BTU/lb

   and $3.00/10(6) BTU = $27.00/mg

   Total Fixed Costs = $81.00/mg

Variable Costs:

Apply 0.5 economy of scale based on carbon dosage:

ratio to BIO AWT = 1800(0.5)/250 = $2.60

The results of this computation are shown on Figure II-14, with the original costs from Technical and Economic Review of Advanced Waste Treatment Systems, and the costs after index adjustment for the Bio-based AWT.

Breakpoint chlorination.

Breakpoint chlorination O&M costs are virtually all (85-90 percent) due to the purchase price of chlorine (P/C Nitrogen Removal, EPA, 1974). Chemical suppliers quoted a price of 6.7 cents per pound of chlorine gas, delivered, for large quantities in the Study Area. Total O&M costs were computed from a design dosage rate of 10 mg/l chlorine per 1 mg/l ammonia nitrogen, and the B-JC average ammonia nitrogen of 0.03 lb/cap/day, primary effluent, i.e., a chlorine dosage of 0.30 lbs/cap/day. Thus, per capita chlorine cost is 2 cents per day, or $7.30 per year; total O&M was computed by adding 15 percent to the chlorine cost, for a total of $8.40 per capita per year. For each service area, breakpoint chlorination costs were computed, based on Stage II-2 design population and these costs plotted against flow (Figure II-15).

Land Treatment of Secondary Effluent.

There was little experienced information on which to base estimates of land application O&M costs, since there is no operational record with the characteristics of the systems designed for this study (size, 20 to 30 mgd; hilly topography; and use of both forest and open agricultural land). Therefore, the method used to estimate O&M costs was to make original estimates of these costs and to compare these costs for order of magnitude with published information. Estimates were made for two basic cost items, power and labor. Since power costs can be predicted fairly precisely, and since these will vary markedly depending on topography (pumping head), only the labor costs were compared to available literature information.

Pumping power costs were based on the preliminary design of pumps, the study area power rate structure ("Electric Rates for General Service," 1974), and a preliminary operating schedule.

The following elements were included in the estimate of power costs:

FIGURE II - 14

ACTIVATED CARBON O & M

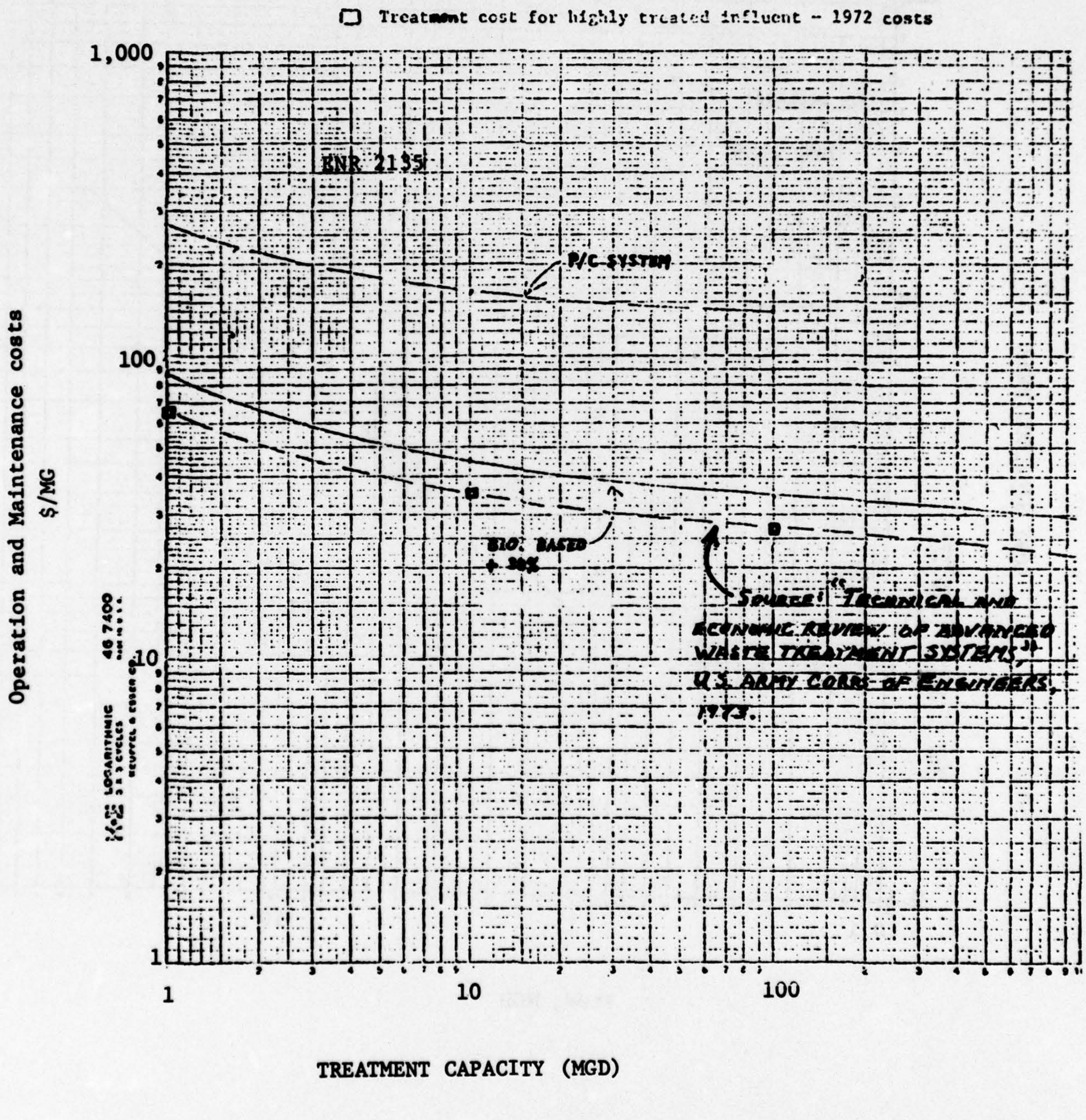
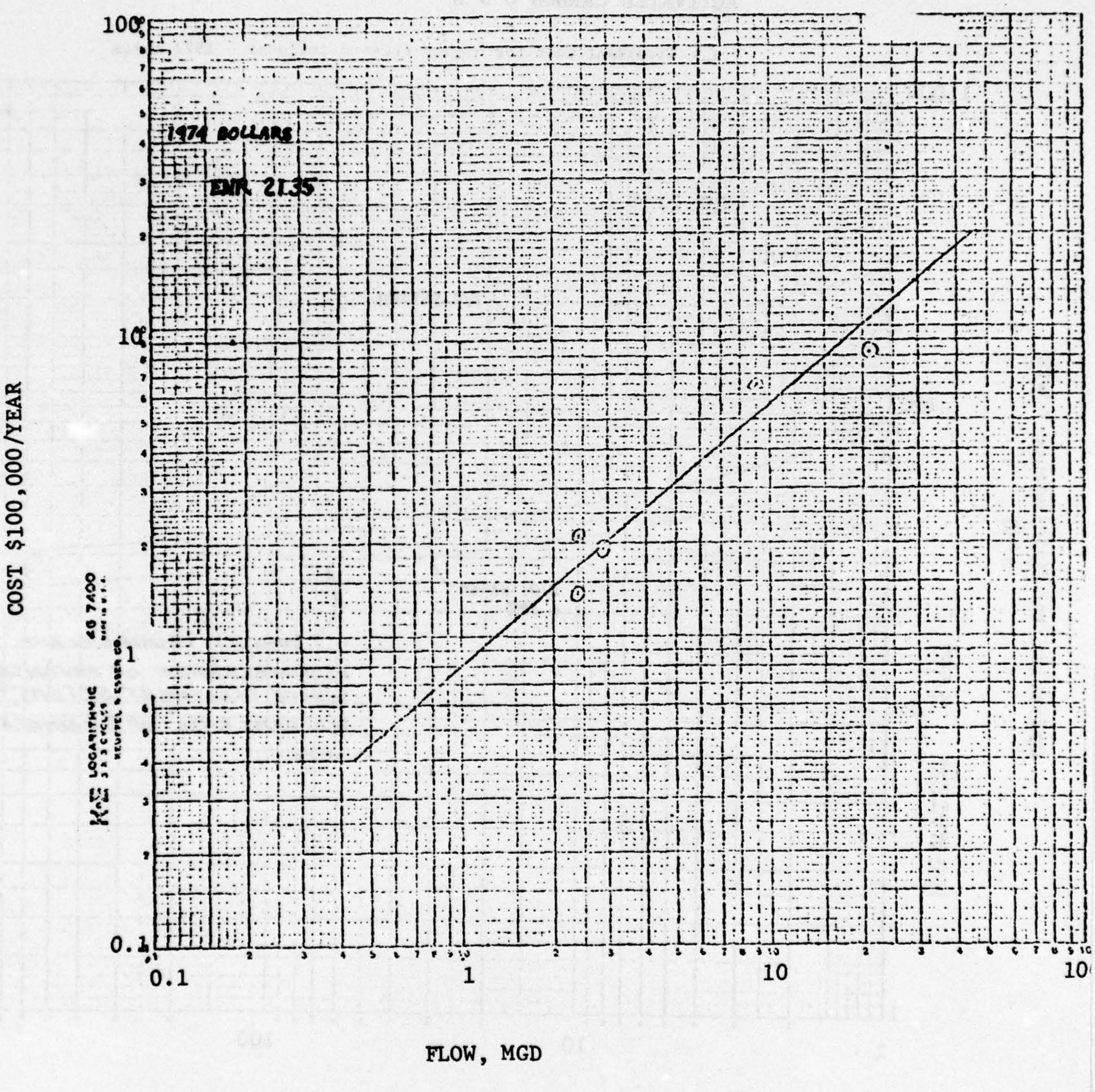


FIGURE II - 15

BREAKPOINT CHLORINATION O & M



1. Lift Station from STP to Land Application Site Storage:

   a. Hours of operation: 24 hours per day.

   b. Flow (Capacity): 1.5 x average.

   c. Head: Static + Dynamic

   d. Demand charge based on capacity.

   e. Consumption charge based on average flow.

   f. Special demand charge for idle time (six months).

2. Lift Station at Application Site:

   a. Hours of operation: 6 hours per day.

   b. Flow (Capacity): Average Flow.

   c. Head: Static (Calculated lift plus nozzle pressure) + dynamic

   d. Demand charge and consumption charge on average flow.

   e. Special demand charge for idle time (six months).

Cost computations in Stage III-1 were done in detail for the large system serving the B-JC, Endicott, and Chenango Valley Service Areas, with the result that total power costs are equal to 2.02 cents per KWH. Other systems were estimated at 2 cents per KWH.

Labor includes requirements for operation, maintenance, laboratory, and administration. Operational labor is required to perform functions such as pump station operation, valve operation, flow rate checking, and nozzle inspection. At the design application rate of 1.7 inches per week, with a 20 percent buffer zone, 180 acres would be required per 1 mgd of flow. It was estimated that one operating person can handle a site of this size, on a seven-day-a-week basis, and that, for a total application period of six months, that the person would be hired for 7.5 months, to perform functions of equipment preparation and shut-down. It was further assumed that the labor costs for other functions, i.e., pump operation, plus maintenance and laboratory, would be 50 percent of the direct operational costs. Total labor costs per mgd were:

Operation:	(7.5 months x 30.4 days/month x 8 hours/day x \$6/hr)	\$11,000/yr
Other:	50%	5,500
Total:		\$16,500/mgd

To this cost was added \$2,500 for materials, for a total of \$19,000 per mgd for all land application O&M costs, excluding power. For a 180-day season, cost is \$105/MG, or \$0.105 per 1000 gallons. This figure was reasonably close to information found in Land Treatment of Municipal Sewage, Costs of Wastewater Treatment by Land Application, and the Codorus Creek Wastewater Management Study.

PHASING OF SYSTEM COMPONENTS

A determination was made of the economical design lives of wastewater collection and treatment facilities. The economic design life is the number of years a structure is planned to accommodate projected loads. Long design periods result in excessive capacity, which is a waste of money; whereas short design lives require continuous planning, design and construction, with a possible loss of economies of scale.

Various authors (Manne, 1967; Richford, 1964; Smith and Eilers, 1971) have shown that with a linearly increasing demand, the optional design period for various components of a wastewater system with time is provided by solving the following equation:

$$a = \frac{r \cdot t}{e^{r \cdot t} - 1}$$

where: r = interest rate, in this case (Stage II-2), r = 5 7/8%

t = optimal design period

a = economy of scale parameter in the general cost function: $cc = k \cdot Q^a$

where: cc = cost
k = constant
Q = demand

The time between expansions in the first equation is independent of the size of the demand and the rate of increase in demand.

The optimal design period is the time at which the present worth of all expenditures is a minimum. The form of the equation giving the present worth of a series of plant expansions is:

$$PW(D \cdot t) = k(D \cdot t)^{a} \quad (1 - e^{-rt})^{-1}$$

where: D = linear demand function

$$Q(t) + D \cdot t$$

The impacts of the variations of "a" on the present worth are illustrated in Figure II-16. Additionally, the sensitivity of the present worth to changes in the optimal design period is demonstrated to the 10 percent level by the shaded areas along each curve.

The assumptions made in developing Figure II-16 were:

1. Linearly increasing demand.

2. O&M costs are fully 100 percent flow variable, i.e., O&M completely dependent on actual flow and not capacity).

3. O&M cost is small in relation to construction costs if not fully flow variable.

The projections of future flows show that loads will generally increase linearly for the next 10 to 20 years, and then exhibit a rapidly declining growth rate. Declining growth rates result in longer design periods. Therefore, the plans generally provide for short design periods for early construction, and longer design periods for construction well in the future.

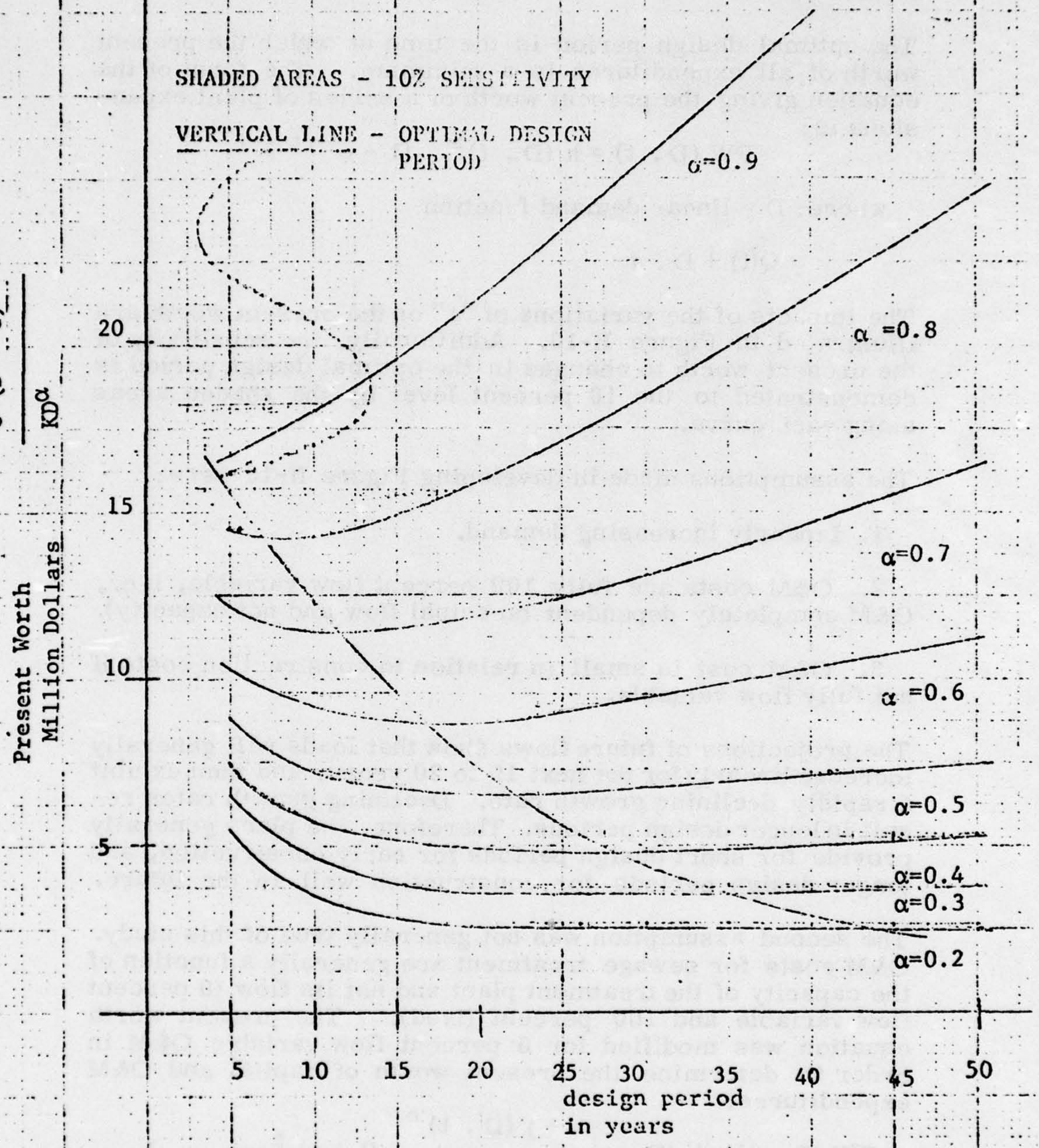
The second assumption was not generally true of this study. O&M costs for sewage treatment are generally a function of the capacity of the treatment plant and not its flow (0 percent flow variable and 100 percent fixed). The present worth equation was modified for 0 percent flow variable O&M in order to determine the present worth of capital and O&M expenditures:

$$PW(D \cdot t) = k(D \cdot t)^{a + j(D \cdot t)^{b}} \quad \frac{(1 + r)^{t} - 1}{r(1+r)^{t}} \quad \frac{1}{1-e^{-rt}}$$

where "j" and "b" are the parameters of the O&M cost equation: $OM = j \cdot Q(t)^{b}$. The equation can be solved for the

FIGURE II-16

Present Worth vs. Design Period
(O & M 100% Flow Variable)



- for generalized capital only
- for family of α (economy of scale) curves of the form $cc = k\cdot(D\cdot t)^{\alpha}$
- for O & M 100% flow variable
- for linear demand growth

economic design life by differentiating both sides of the equation with respect to time. The solution yields a rather complex expression with one important feature: the design period is dependent on the value of "D", the demand function. This finding is significant because the demands on each plant are different. Therefore, each treatment plant should be considered separately when facilities plans are written for future construction.

In order to test the importance of O&M in design period determination, a trial and error determination was made for the case of the proposed Chenango Valley STP. The construction cost (from capital cost curve used in Stage II-1) are estimated as:

$$cc = 1.239\,(D \cdot t)^{0.611}$$

where: D = 0.16 mgd/year.

The O&M costs (from O&M cost curve used in Stage II-1) are estimated for two cases:

1. The O&M is 0 percent flow variable:

   $$O\&M = 49{,}407\,(D \cdot t)^{0.776} \quad \text{dollars.}$$

2. O&M is 50 percent variable (a time function):

   $$O\&M\,(t_o) = 50\% \times 49{,}407\,(D \cdot t)^{0.776} \quad 1 + t_o/t$$

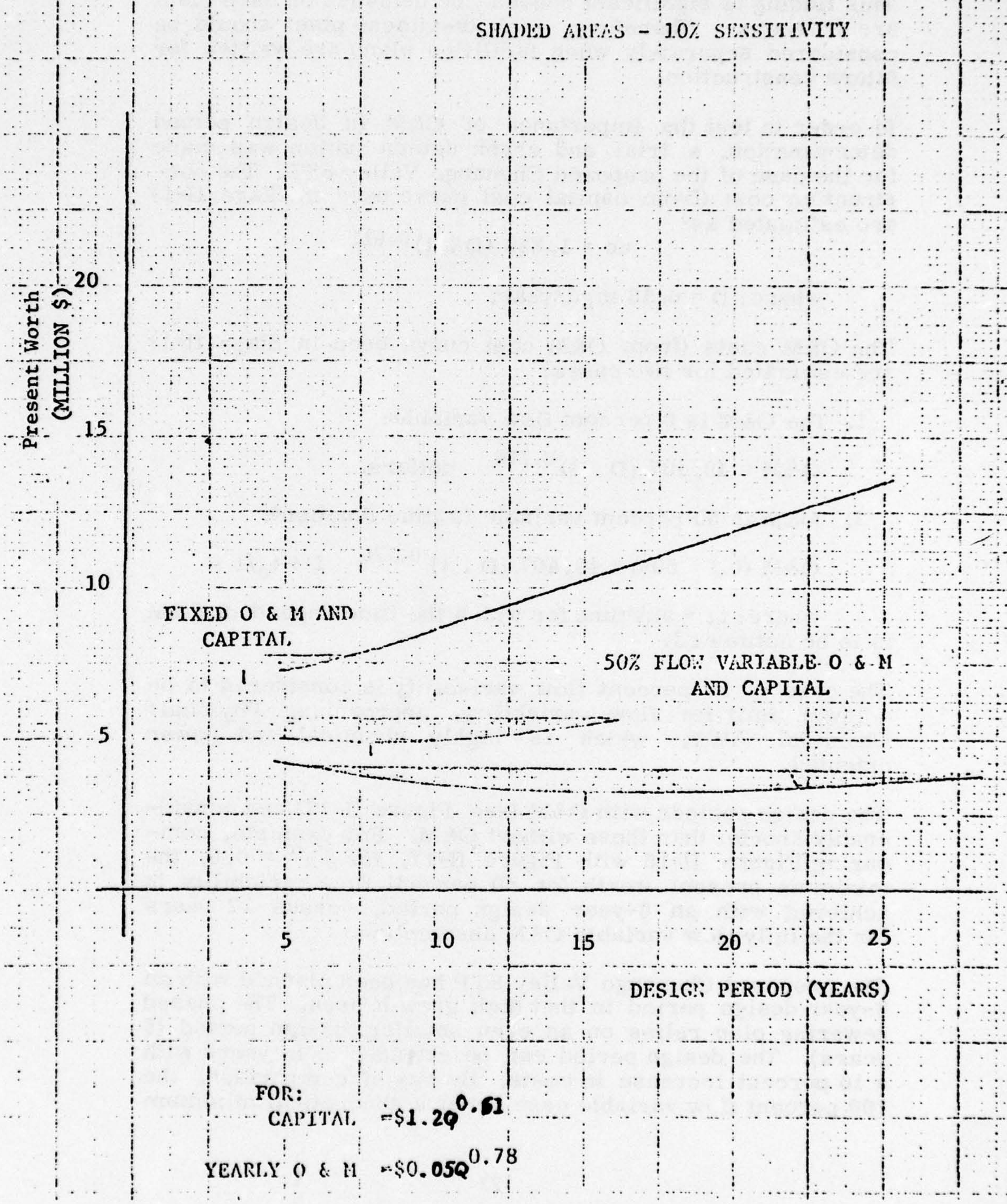
   where: t_o = any time for which the time dependent O&M is to be estimated.

The case of 50 percent flow variability is considered to be a lower limit for flow variability, approaching Physical/Chemical AWT, which is highly chemical and power oriented.

The design periods with O&M (see Figure II-17) are considerably shorter than those without O&M. For example, comparing Figure II-16 with Figure II-17, for "a" = 0.6, the minimum present worth for 50 percent flow variability is achieved with an 8-year design period, versus 17 years for the fully flow variable O&M assumption.

The proposed Chenango Valley STP has been planned with an 8-year design period in that high growth area. The phased sewering plan relies on an even smaller design period (5 years). The design period can be extended to 16 years with a 10 percent increase in costs. By way of comparison, the 100 percent flow variable case noted a theoretical minimum

Figure II- 17
PRESENT WORTH vs DESIGN PERIOD



around 17 years. The question then becomes: "How valid is the use of an 8-year design period for municipal waste treatment?"

With the exception of a possible Chenango Valley STP, a minimum design period of 10 years and a general maximum governed by the 10 percent sensitivity of 16 years was used through Stage II-2 of this Study. The use of a range of 10 to 16 years would allow for a consolidation of construction at treatment plants, especially with respect to phasing of AWT add-ons.

In Stage III, the design lives were changed to reflect such factors as the reduced population and flow projections. The use of lower projected flows tended to increase the design life of treatment components as wasteful overdesign would not be as critical a factor. In addition, the use of design lives greater than the theoretical economic were justified by:

1. Problems of fund raising in respect to:

   a. Possible referendum.

   b. State and Federal priorities.

2. Problems of planning with respect to:

   a. Length of time for planning to be initiated.

   b. Length of time necessary for planning and design.

   c. Length of time for final plans to be accepted.

3. Problems of construction at treatment plants with respect to:

   a. Interference with personnel.

   b. Interference with general plant operations.

   c. Possible system failures during construction and testing.

4. Problems of construction of interceptors with respect to:

   a. Traffic disruption.

   b. Commercial area impacts.

   c. Noise, dust.

There is a further consideration with respect to STP expansion, which would favor design lives either greater or less than the economical optimum, i.e., the concept of modular expansion of existing treatment units. Modular expansion is almost a necessity with rectangular, common-wall construction, as in aeration tanks and rectangular clarifiers. Even with circular units, however, modular expansions greatly simplify flow distribution to parallel units such as trickling filters, circular clarifiers, and sludge thickeners. In the design of alternatives, therefore, all rectangular STP expansion units were designed to be modular with those presently existing. For circular units, expansions were modular except in cases where the resulting capacities would represent large over-designs. Table II-15 is a summary of the design lives used for major capital items:

TABLE II-15

DESIGN LIVES FOR MAJOR CAPITAL ITEMS

	Range, Years
Pump Stations	13--50
Interceptors	50
Secondary STP (New)	5--8
STP's (Expansions of Secondary)	13--50
STP's (Nitrification)	27--44
STP's (AWT)	7--42

Design lives at the lower end of the range generally reflect areas with more rapidly changing conditions, such as projected flow increases. Those at the upper end are indicative of more stable conditions. The design period for interceptors was 50 years, slightly larger than that allowed by the 10 percent sensitivity. The design periods greater than the theoretical were used, because interceptor construction entails disruptions which justify a design life virtually equal to their useful life.

In summary, design lives greater than the theoretical economic were generally used in order to reduce disruptions and the need for continuously raising new funds for capital investments for capacity expansions. These expansions would be required at the end of the design life of the previously constructed or existing items, and can be designed according to the principles established in the prior discussion.

REPLACEMENT

Replacement costs will be encountered throughout the planning period, as structures and mechancial equipment reach the end of their useful lives. These costs were considered in the comparative evaluation of alternatives, in order to investigate fully the economic implications of investment decisions made at the outset of the planning period.

Various types of capital items have different useful lives, as indicated by Table II-16, from the EPA 208 Guidelines:

TABLE II-16

USEFUL LIVES OF WASTEWATER TREATMENT COMPONENTS

Item	Useful Live, Years
Land	Permanent
Structures	30 -- 50
Process Equipment	15 -- 30
Auxiliary Equipment	10 -- 15

Separate evaluation of each of these items, however, would lead to extremely complex and detailed analyses, and is to be done under Step 2 of the EPA program (detailed design and specifications).

In order to overcome the complexities associated with the separate items, the concept of a composite useful life was introduced into the economic analyses. The composite useful life of a capital installation was defined as that life which yielded a present worth of replacement of the entire capital installation equal to the sum of the present worths of replacement of the component parts at the end of their individual useful lives. Thus, using the lives of individual components as given in the EPA Guidelines, and the known percentages these individual costs are of the total cost of an installation, the composite useful life was calculated.

Ignoring land costs, which are usually comparatively small in STP construction, the following breakdown was found representative of the useful lives of the three major capital cost components: Structures--50 years; process equipment--25 years; and auxilliary equipment--15 years. Using this breakdown and the interest rate used in Stage II-2

(5 7/8 percent), it was found that the composite life of sewage treatment plants was 27 years. Although no guidelines were available on the useful lives of pump stations, it is known that these are similar to treatment plants, so that a 27-year composite life was also used for pump stations.

Pipelines, including interceptors and force mains, have useful lives well over 50 years, i.e., beyond the design period. In the economic analysis, therefore, no replacement of these items was assumed.

The land application system consisted of some equipment (main transmission pipe), which would accumulate sufficient capital, at compound interest, to replace each major capital expenditure at the end of its useful life.

Since replacement goes on continuously, the sinking fund was scheduled to the end of the fifty year period of analysis. The sinking funds or replacement costs for the existing facilities were based on the estimated invested cost for each treatment plant in 1975 dollars (adjusted to ENR 2248) as shown in Table II-17.

TABLE II-17

REPLACEMENT COSTS OF EXISTING PLANTS
(Existing investment updated to 1975 $)

Binghamton-Johnson City	$21 Million
Endicott	$15 "
Owego WPCP #2	$3.0 "
Owego WPCP #1	$0.55 "
Owego Village	$0.8 "

For new facilities designed for the various action plans, the sinking funds match the initial expenditures at the end of their useful lives.

PRESENT WORTH

This study used a present worth analysis to evaluate the various costs of the alternatives, based on methodologies described in <u>Guidance for Preparing a Facility Plan</u>, EPA, (1975), and <u>Principles of Engineering Economy</u>, Grant and

Ireson, (1964). Interest rates used have been previously given; the period of economic analysis was 50 years.

The first year of analysis is 1977. The alternatives are defined such that all proposed construction begins no sooner than the beginning of year 1977. Although the actual construction at different facilities might begin at different times, due to constraints of detailed project planning and implementation, all initial construction is scheduled for the beginning of year 1977. This assumption was made for purposes of comparison in a cost-effectiveness determination.

Sunk costs (existing debt service), though important to consider in cost-sharing programs, were ignored in calculating present worths (as dictated by theory and Federal regulations).

Only direct costs for construction of interceptors and treatment facilities, rehabilitation of interceptors (infiltration control), inflow control and yearly operation and maintenance costs at treatment facilities are included in the analysis, i.e., local sewer system construction and maintenance costs are not included. In addition to construction and O&M costs, replacement costs (as discussed in the preceding section) are included as a yearly sinking fund for existing treatment faiclities and new treatment construction.

Costs for construction are assumed to be beginning of year payments, whereas O&M and replacement costs are assumed to be end of year payments. So, 1977 construction means that payment will be made for the construction on 1 January 1977. An O&M cost in the same year requires payment on 31 December 1977.

Zero salvage values were assumed for the various facilities at the end of the 50 year period. The influence on the total present worth of not including a salvage value at the end of the 50 year period of analysis was negligible (about 2 percent). a computer program was written to calculate present worths.

AVERAGE ANNUAL COST

The average annual cost of an alternative is that amount of money expended every year for 50 years, which has a present worth equal to the present worth of all alternative expenditures, at an equal interest rate. For purposes of plan

comparison, average annual cost provides an easily computed approximation of actual expected annual costs which, in fact, will vary somewhat year to year due to future installations.

PER CAPITA AVERAGE ANNUAL COSTS

The cash flow of year by year local expanditures were analyzed for each alternative. The computer program was used to analyze the cost to the individual services by a treatment plant and was therefore used most heavily in the Impact Assessment and Evaluation Appendix.

Computer Program Development

The program was written in FORTRAN IV. Storage constraints on the IBM System 3 (Lawler, Matusky, and Skelly Engineers) necessitated program dimensioning to exactly tailor the input data. Additionally, changes during the conduct of the study required internal changes to the program (to adjust for a cost index change, to alter industrial flow projections, and to manipulate certain input data). The development and output of the computer program is discussed below.

1. Outside Grants. The program can analyze any level of State and Federal construction grant. Additionally, any level of operaation and maintenance grant can be analyzed.

2. Costs. Three types of costs were analyzed: construction, operation and maintenance, and replacement. The program can be run with or without replacement costs. If the program was run with replacement costs, a sinking fund was constructed internally within the program for all capital items. The technique is the same as that for the cost effective analysis sinking fund. However, no grants other than the 50 percent of recovered Federal grants attributed to industry (see Section 6 Below), were used to subsidize this fund. If replacement was not to be considered as a cost, the sinking fund and the 50 percent of recovered Federal grants attributed to industry were omitted from the cash flow. The program can analyze linearly increasing O&M payments between any two years.

3. Debt Service. A major feature of the program is that a debt service schedule which conforms to the requirements of the New York State Local Finance Law, which requires that "no annual installment (of principal) shall be 50% in excess of the smallest prior installment." This restriction results in total debt service payments which decline with time.

Since existing debt service for treatment plants is a sunk cost and not an attribute of any alternative, such case flows were not included in the alternative analysis (O&M for existing plants was included, however).

4. Flows. Total flows and populations are input to the program at the wastewater management area level for benchmark years. Infiltration flows were input at the wastewater management area level. Additionally, uniform (for the entire region) per capita residential flows and per capita residential and service flows were input for each benchmark year. The program calculated the total per capita flows at each benchmark year by dividing the total flow less infiltration by the population.

Between benchmark years, the flows for each year were derived by a straight line interpolation and when the period of analysis went beyond the last benchmark year (2020), straight line extrapolation predicted the yearly flow and population. Sewering was always assumed to be 100 percent of the available flow.

Once the alternatives were defined, infiltration-inflow reductions for each treatment plant were input to the program.

5. Cost Allocation and Recovery. This analysis assumed the costs of wastewater management were allocated 100 percent on the basis of volumetric flow. All costs were initially allocated to the following categories: Residential, Service Industry, Other Industry, Infiltration, and Excess Capacity.

According to Federal regulations, the portions of Federal construction grants allocated to industry are to be recovered from industry over a 30 year period without interest. Therefore the percentage of industry to the total flow times the Federal grant was the amount charged to those industries.

Although the regulations provide that industries which only discharge normal domestic wastes need not be charged for Federal recovery, the analysis assumes full recovery from all nondomestic users (ignoring government).

The charges to infiltration and excess capacity are paid according to flow by the three other classes of users. Since excess capacity and infiltration are not flows directly attributable to industry, no Federal grant recovery is levied against the two. The analysis assumed no excess capacity would be specifically reserved for any user. Since excess capacity changes from year to year, the computation for industrial recovery was made for each year.

The charges were based on the cash necessary to retire bonds and pay for O&M, less any state construction or O&M grants, for which there is no cost recovery requirements. Additionally, the charge to residential users is based on the treatment plant service level and not wastewater management areas or other division. In other words, all costs were shared equally by all users (except as noted above).

6. Recovered Federal Grants. Regulations stipulate that 50 percent of the recovered funds from industry is to be returned to the government. Of the other 50 percent, 80 percent must be used for the sinking fund. If the sinking fund analysis is included, all of the 50 percent was applied to the sinking fund account before the account is allocated to each user. If the sinking fund analysis is excluded, all of the recovered local costs were also ignored. In the latter case, however, the amount returned to the Federal government was still summarized.

7. The output included summary cash flow tables which provided the year-by-year payments of all local expenditures (excluding the 50 percent of recovered industrial funds sent back to the Federal government), the charges to residential users, the per capita charge to service and residences, all charges to other industry, the amount of Federal construction grant (if construction takes place in a particular year), the amount of money returned to the Federal government, the amount of State construction grant, and the yearly State O&M grant.

Inputs for the Urban Study Area

The maximum levels of State and Federal aid were assumed for the analysis total to be 12.5 percent and 75 percent, respectively. The State O&M grant was assumed to remain at 33 percent of eligible costs. All future construction was assumed to be covered by the existing grant levels. Future replacement costs were assumed not to be eligible for outside grants.

As previously discussed in this Chapter, there were two flows investigated based on (1) an increasing per capita flow rate, and (2) the continuance of the 1973 per capita rate as a result of nonstructural flow reduction measures. The initial residential and residential & commercial per capita flows were assumed to be 60 and 80 gpcd, respectively, in 1973.

The assumptions were refined in Stage III-2 for the West Owego and Endicott service areas to more accurately reflect the actual service activity. The service (commercial) flow limits, including the per-decade rate increases, were changed to 94, 96, 84, and 98 gpcd for year 1973, 1980, 2000, and 2020, respectively, in the West Owego WWMA. The limits for the Endicott residential and commercial rates were decreased by 8 percent and 15 percent, respectively.

The above assumptions were made as a preliminary investigation into a cash-flow analysis of per capita annual costs (for use in impact assessment).

Although sensitivity runs were made, the following was assumed for the final use of the program:

--inflation of 0 percent.
--income rise of 0 percent.
--municipal bonding rate of 6 1/8% with a period of 30 years.
--evaluation interest rate of 6 1/8%.
--period of economic analysis of 50 years.

SUMMARY

This chapter has dealt with the basic elements of the design and economic analyses, to document the overall analytical framework. The chapter has included some aspects specific to the Study Area, such as population and wastewater flow projection, but primarily has dealt with explaining the methodology of the cost and economic analysis, less specific to the study area. Most of the items discussed would have been approached in a similar manner even in a completely different physical setting.

The existing physical setting and the expected 1977 Baseline Condition are described in Chapter III. Chapter IV discusses the overall considerations, employed on the design of alternatives, and Chapters V, VI, and VII present the details of the major technological analyses used to develop the wastewater, sludge and stormwater management components, respectively. The final plans are discussed in Chapter VIII.

CHAPTER III

EXISTING AND 1977 BASELINE CONDITIONS

This chapter is basically concerned with the existing physical wastewater management systems, and proposed local plans for installations expected to be implemented in the near future (by 1977). This information was incorporated into both the Baseline Plan, discussed below, and the design of each action alternative.

The discussion of existing wastewater management physical systems will be in summary form, since the detailed analyses of items such as the capacities of the Binghamton--Johnson City STP and the Endicott STP are discussed in Chapter V.

EXISTING WASTEWATER TREATMENT

The five existing sewage treatment plants considered for continued use throughout the planning period are: Binghamton-Johnson City, Endicott, East Owego (Town of Owego STP No. 2), West Owego (Town of Owego STP No. 1), and Owego Village.

In addition to these STP's, there is a 1.0 mgd primary plant in the Town of Vestal. This plant will be replaced in the near future by a pump station which will discharge the Vestal service area wastes to the Endicott STP for treatment, and therefore, is not described herein. Also not included is the Valley View Imhoff Tank (0.05 mgd) in Owego Village. Wastewater from Valley View will be sent to the Owego Village STP in the near future. The existing STP's and their

service areas are shown in Figure III-1. At present, there are no sewers or sewage treatment plants in the Chenango Valley service area. Alternatives which treat the Chenango Valley wastes separately require a new STP in that area.

The service areas, present and projected populations and flows for the various service areas have been summarized in Chapter II. Following is a discussion of the basic conclusions drawn with regard to these plants which influenced the definition of alternatives in Phase II. Generally speaking, this discussion centers on potential limitations of STP expansion or modification to fit the requirements of the various alternatives.

Changing roles for each STP were dictated by:

1. Increases in flow due to increases in service area, per capita flow, degree of regionalization and by one particular method of stormwater overflow control (storage and subsequent discharge to the B-JC STP);

2. Decreases in flow due to infiltration control;

3. Increases in treatment efficiency needed to meet either water quality objectives or specific treatment objectives, e.g., secondary treatment or AWT;

4. Changes in sludge handlling methods to accomplish objectives of recycle, economy, or environmental quality;

5. Compatibility with land application of liquid effluent.

The factors involved in determining if a given STP could adequately respond to these changing roles included present flow rate; present method of treatment, efficiency and capacity; land area available for expansion; and location of discharge. The ability of each plant to meet these conditions is briefly discussed.

BINGHAMTON-JOHNSON CITY STP

The basic descriptive information on the Binghamton-Johnson City STP is given in Table III-1. This treatment plant was pivotal to wastewater management planning in the study area for three major reasons:

SEWAGE TREATMENT PLANTS AND THEIR SERVICE AREAS

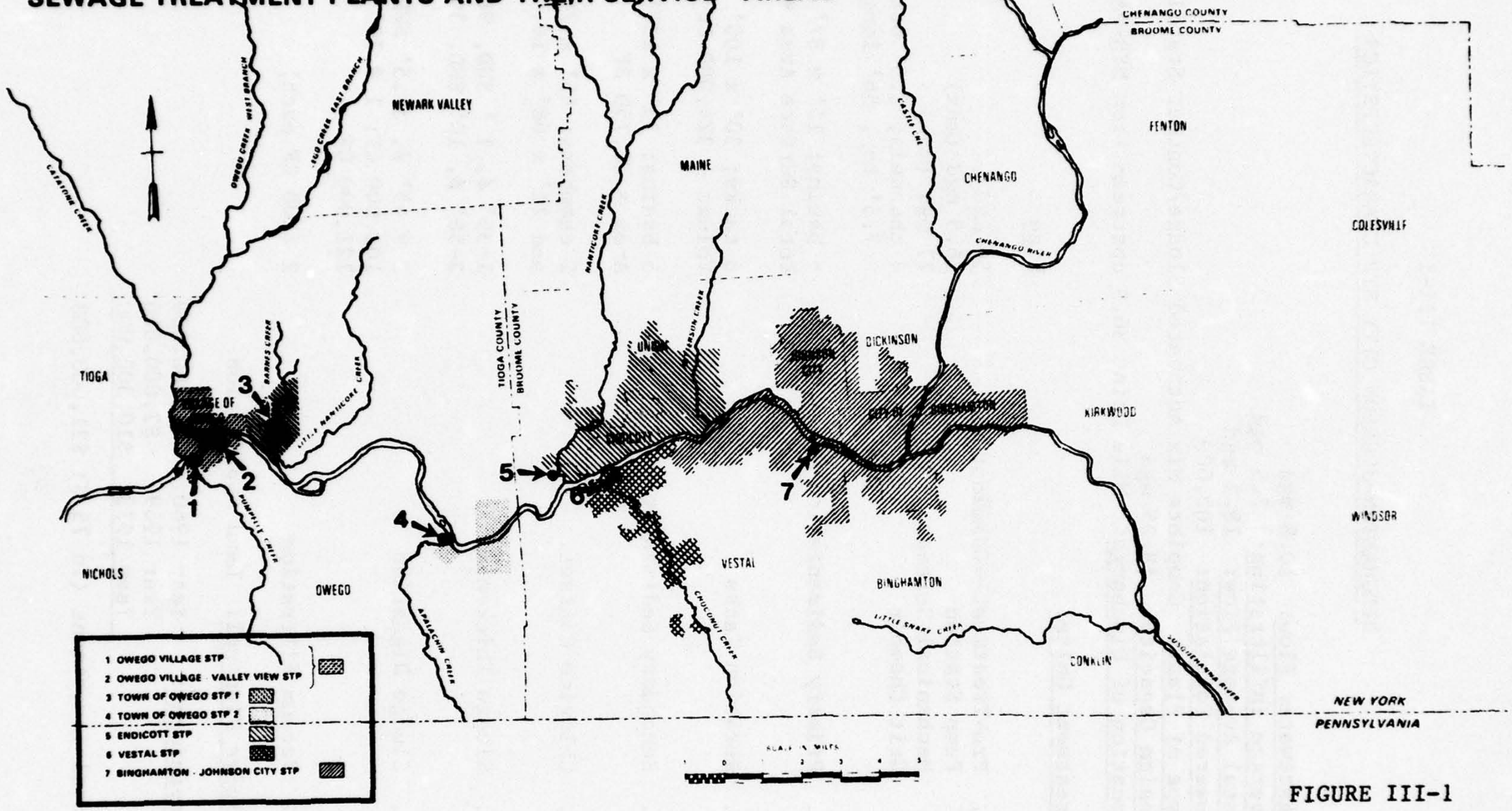


FIGURE III-1

TABLE III-1

BINGHAMTON-JOHNSON CITY STP CHARACTERISTICS

Wastewater Flow: 10.8 mgd
Average Infiltration: 7.5 mgd
Total Average Flow: 18.3 mgd
Sewered Population: 102,000
Type of Plant: Complete Mix Activated Sludge/Contact Stabilization
Design Capacity: 18.25 mgd
Location of Discharge: Mile Point 36.9 upstream from NYS-PA State Line.

Treatment Units:

	Size
1. Pre-Treatment--Commdinutors	50 mgd
Pump Station	36.5 mgd (max)
Mechanical Screens	27 mgd (max)
Grit Chamber	4 channels; 2.5' bottom, 7.5' top, 56' long
2. Primary Sedimentation	6 basins; 15' x 87' x 12' depth. Total Surface Area = 7,830 SF
3. Aeration Tanks	6 tanks; 30' x 100' x 18' depth. Volume - 324,000 CF
4. Secondary Sedimentation	5 basins; 33' x 138' x 11-12' depth. Area = 22,750 SF
5. Chlorine Contact	2 chambers; 45' x 78' x 6.2' depth. and 21' x 68' x 16.4' depth.
6. Sludge Thickening	1-35' ϕ, 1 ' SWD, 960 SF: 2-50' ϕ, 10' SWD, 3920 SF/2 units.
7. Sludge Digestion	2 @ 45' ϕ, 31.5' SWD, VOL/2 units = 100,000 CF; 1 @ 70' ϕ, 31' SWD, 121,000 CF
8. Vacuum Filtration	2 (200 SF each)

Sludge Disposal: Land Application

Investment:	Year 1960	$2,000,000
	Year 1969	$2,000,000
	Year 1973	$10,300,000
Total Investment (in 75 $)		$21,000,000

1. It is by far the largest plant, and, therefore, has the greatest effect on Susquehanna River dissolved oxygen (DO). Alternatives to meet target levels of DO must consider increased oxygen demand removal at this plant before any others.

2. It is fed by a combined sewer system subject to storm water overflows. Therefore, in the stormwater management alternative for overflow storage and subsequent discharge to the STP, its hydraulic and organic load would be increased, on an annual basis, over that prevailing in other stormwater management alternatives, i.e., treatment and discharge at the overflow point.

3. The most controversial questions with regard to regionalization of facilities, i.e., to treat the Chenango Valley area's wastes separately or at the B-JC STP, revolved to a certain extent around the capability of this plant to treat the extra flow from this area.

Before this study began, the question of the capacity of this plant was already under discussion by officials of NYSDEC and local officials. This discussion resulted primarily from the fact that, although its aeration detention time is short (3.2 hours at design average flow of 18.3 mgd, and 3.4 hours at summer 1974 flow at 17 mgd), it achieved high degrees (greater than 90 percent) of BOD removal and nitrification in the summer months of 1973 and 1974. It was therefore felt that the plant could be rated at a higher capacity than 18.3 mgd.

An analysis was conducted of the plant design and performance in detail, to determine its capabilities with respect to the Alternatives, and to enable specific expansion process designs for these Alternatives. The analyses and design are discussed in detail in Chapter V. The conclusions reached were as follows:

1. The B-JC STP is not overdesigned with respect to secondary treatment capacity, and will need expansion of these facilities (aeration and clarification tankage) for any Alternative which results in flow increases to this STP.

2. Alternatives which include the Chenango Valley Service Area wastes could require as many as three new aeration and clarification tanks, which is the limit of the present site for expansion.

3. AWT facilities could be accommodated on the site, but again, the fit is tight, and may require modification of plant piping.

4. For the alternative of stormwater storage for subsequent discharge to the plant, the plant as designed for each dry weather flow scheme does have the required secondary treatment capacity, even though the flows after a storm will be higher. The reason for this is that the treatment plant design capacity is controlled by winter conditions of low temperature, so that the plant would be overdesigned for summer conditions, when the design storm occurs.

To summarize the above conclusions, the existing B-JC STP could be modified to meet the requirements of any of the alternatives. It is however, on a small site, and major expansions, particularly for AWT facilities would create difficulties.

ENDICOTT SEWAGE TREATMENT PLANT

This STP is presently loaded well under its design capacity, i.e., a present flow of 4.2 mgd versus its design capacity of 7.7 mgd (secondary) and 12.5 mgd (primary). Even with the connection of the present Vestal STP's wastes to this plant, approximately 1.0 mgd, the Endicott STP will have sufficient capacity until about 1983. Basic information on this plant is summarized in Table III-2.

The operating records of this STP were analyzed with the intention of establishing a design capacity and expansion requirements, for the purposes of this study. This analysis (see Chapter V) was hindered by the poor performance of the plant during the Study, apparently due to a toxic substance in the plant influent. For this reason, standard design formulae were relied upon.

A review of standard design procedures revealed that it is quite possible that trickling filter plants in the Study Area climate will not (in the winter) meet the secondary treatment criteria of a minimum of 85 percent BOD removal and a maximum effluent BOD of 30 mg/l. Regulatory officials doubt that well operated trickling filter plants, such as Endicott's, can meet the standard in the winter. It appears that approximately 82 percent removal will be attained at the Endicott design loading rates.

However, since the difference between 82 and 85 percent BOD removal at Endicott is negligible in terms of Susquehanna River DO, and in the absence of official guidance

TABLE III-2

ENDICOTT STP CHARACTERISTICS

Wastewater Flow: 3.5 mgd
Average Infiltration: 0.7 mgd
Total Average Flow: 4.2 mgd
Sewered Population: 45,000
Type of Plant: Trickling Filter
Design Capacity: Primary--12.5 mgd; Secondary--7.7 mgd.
Location of Discharge: Mile Point 29.4 upstream from NYS-PA State Line

Treatment Units:

		Size
1.	Preliminary Treatment	
	Raw Waste Pumps	31.6 mgd (max)
	Comminutors	2 @ 15.8 mgd = 31.6 mgd
	Grit Chamber	34 mgd
2.	Primary Sedimentation	3 @ 32' x 173' x 8'10" Hydraulic Capacity = 12.5 mgd (aug)
3.	Trickling Filters	2 @ 120' ϕ x 6' depth; Volume = 136,000 CF
4.	Secondary Settling	2 @ 80' ϕ x 8' SWD; surface settling rate=765 gpd/SF ea; Avg Flow=3.83 mgd ea.
5.	Chlorination	31.6 mgd (including outfall)
6.	Sludge Thickening	2 @ 35' ϕ x 10' SWD: Area = 1,924 SF
7.	Sludge Digestion	2 @ 50' ϕ x 27.8' SWD; Volume = 109,000 CF
8.	Vacuum Filtration	2 @ 440 SF = 880 SF

Sludge Disposal: On-side Landfill

Investment:	Year 1966	$4,200,000
	Year 1972	$4,200,000

Total Investment (in 1975 $) $15,000,000

from the regulatory agencies, it was decided for the purposes of this study that the trickling filter process would meet secondary treatment requirements at Endicott.

There is ample land area at this site to accommodate any of the treatment units (including AWT) required in any of the alternatives for all flows expected in the planning period.

OWEGO #2 STP (East Owego)

The Owego No. 2 is an activated sludge secondary treatment plant. The present flow of 0.4 mgd is well under the design capacity of 2.0 mgd (Table IIII-3). Alternatives for this plant, in addition to considering varying degrees of treatment, also included the concept of treating the wastes from the Owego Village and West Owego service areas at the East Owego STP. This regionalization scheme was rejected at the end of Stage II-2, however, for environmental reasons. There would be no problem in accommodating treatment units required for BIO AWT, the largest land user, throughout the planning period.

OWEGO #1 STP (WEST OWEGO)

The Town of Owego STP No. 1 is a secondary treatment plant (trickling filter) with a design capacity of 0.5 mgd. Of the total present flow of 0.2 mgd, IBM contributes 0.14 mgd of mainly sanitary wastes. Characteristics of this STP are given in Table III-4. Alternatives for this plant included abandonment and treatment of its service area wastes at either the Owego Village or East Owego STP, and treatment of the Owego Village wastes at this site in addition to continuing to treat only its own service area wastes.

Treatment at the East Owego site was dropped due to environmental reasons. In considering the question of regionalizing the West Owego wastes with the Owego Village wastes, at either plant, secondary and AWT land requirements were considered. There is ample space at the West Owego site for an AWT system large enough to handle both service area's wastes.

TABLE III-3

TOWN OF OWEGO STP #2 (East Owego) CHARACTERISTICS

Present Flow: 0.4 mgd
Sewered Population: 6,500
Type of Plant: Activated Sludge/Contact Stabilization
Design Capacity: 2.0 mgd
Location of Discharge: Mile Point 25.4 upstream from NYS-PA State Line

Treatment Units:

		Size
1.	Preliminary Treatment:	
	Bar Rack	1 @ 5 mgd
	Comminutor	1 @ 5 mgd
	Grit Separators	1 @ 5 mgd
2.	Primary Settling Tanks	2 @ 1,250 SF = 2,500 SF
3.	Aeration Tanks	62,000 CF
4.	Final Settling	2 @ 1,320 SF = 2,640 SF
5.	Chlorination	2 @ 26,000 Gallons = 52,000 Gallons
6.	Sludge Thickening	1 @ 460 SF
7.	Sludge Digestion	2 @ 38,000 CF = 76,000 CF

Sludge Disposal: Truck liquid sludge to land application.

Investment: Year 1972 $2,400,000

Total Investment (in 1975 $) $3,000,000

TABLE III-4

TOWN OF OWEGO STP #1 (West Owego) CHARACTERISTICS

IBM Flow: 0.14 mgd
Total Flow: 0.2 mgd
Sewered Population: Residential--600; IBM--5,000
Type of Plant: Trickling Filter
Design Capacity: 0.50 mgd
Location of Discharge: Mile Point 18.3 upstream from NYS-PA State Line

Treatment Units:

Treatment Units	Size:
1. Preliminary Treatment:	
Bar Rack	1.5 mgd
Grit Chamber	2 @ 1.2 mgd = 2.4 mgd
2. Primary Settling	1 @ 1000 SF
3. Trickling Filter	1, 55' ϕ, 5' deep, 0.28 acre-feet
4. Final Settling	1 @ 960 SF
5. Chlorination	1 tank at 1,690 CF (+4,800 CF, Outfall)
6. Sludge Digestion	1 @ 19,400 CF

Sludge Disposal: Truck liquid sludge to land application.

Investment: Year 1972 $420,000

Total Investment (in 1975 $) $600,000

OWEGO VILLAGE STP

The Owego Village STP, providing only primary treatment, is ranked by NYSDEC for funding to upgrade to secondary capabilities. The characteristics of the present plant are summarized in Table III-5. As discussed above, alternatives for this STP included abandonment and treatment at either the West or East Owego STP's, joint treatment at this site of the Village and West Owego service area wastes, or continued treatment of only the Village service area wastes at this site.

As the plant is presently a primary one, all alternatives except abandonment would require immediate (1977) upgrading to secondary treatment. The present site could accommodate the required treatment units for the wastes from both the Village and the West Owego service areas. For the AWT alternative, however, vacant land adjacent to the present STP would have to be purchased.

SUMMARY OF EXISTING CONDITIONS

To summarize the above discussion, it can be stated that the existing wastewater management facilities can all be expanded or upgraded to serve in the roles demanded of them by the alternatives. Therefore, none of the alternatives were rejected solely on the basis of the existing conditions at any facility.

BASELINE CONDITION

The Baseline Condition formed the basis against which all alternative wastewater management plans were compared. The Baseline Condition was also considered as a possible wastewater management alternative. The physical system characteristics which describe it were those expected to be in existence at the beginning of the planning period (1977). In order to fully evaluate the implications of the Baseline Condition, the assessment of the impacts was performed

TABLE III-5

OWEGO VILLAGE STP CHARACTERISTICS

Wastewater Flow: 0.4 mgd
Infiltration: 0.5 mgd
Total Flow: 0.9 mgd
Sewered Population: 3,600
Type of Plant: Primary
Design Capacity: 1 mgd
Location of Discharge: Mile Point 15.8 upstream from NYS-PA State Line

Treatment Units:

		Size
1.	Preliminary Treatment:	
	Grit Chambers	2, 3'6" x 13'6", 4.4 mgd
	Screening & Comminutors	2.3 mgd
	Raw Sewage Pumping	3.0 mgd max (3 pumps)
2.	Primary Settling	2 tanks; 1,680 SF total
3.	Chlorination	16' x 34' x 6' Volume = 23,500 gallons
4.	Sludge Digestion	1, 30' ϕ x 17; 1, 30' ϕ x 15'6" Volume = 21,614 CF
5.	Sludge Drying Beds	10 beds, 6,219 SF Total

Sludge Disposal: Dried Sludge for public use.

Investment: Year 1966 $400,000

Total Investment (in 1975 $) $800,000

for the year 2020; thus, the Baseline Condition represented the year 2020 inputs (projected development and flows) on the year 1977 physical system, and the resulting impacts.

WASTEWATER MANAGEMENT CHARACTERISTICS

Municipal Wastewaters

It was assumed no sewage treatment plants or expansions would be added other than those approved by NYSDEC for construction in the years 1975 and 1976. Expansion of existing STP's service areas was assumed to continue following the existing trends.

With the exception of the Chenango Valley interceptor, the additional interceptors approved by NYSDEC for funding in 1975 and 1976 were included in the 1977 Baseline Condition.

The Chenango Valley interceptor had been on the NYSDEC priority list, but has since been withdrawn from it, pending a final determination on the question of where this area's wastes will be treated. Those sewers providing service for growth areas included in the Broome County Sewerage Feasibility Study were also assumed to be in the 1977 Baseline Plan. The locations of treatment plants, interceptors and service areas for this plan are depicted on Plate 2.

The following is a description of the facilities in the Baseline Plan that are ranked by NYSDEC for funding prior to 1977.

Owego Village: An interceptor, pump station and a force main to serve all areas south of the river and within the Village; part of River Road and the Valley View Heights subdivision; the area within the Village lying west of the creek; and the low lying area east of the Court Street Bridge, including Lackawanna Avenue and Route 17. The exisitng primary treatment plant will be upgraded to provide for secondary treatment. This project is scheduled for funding in 1976.

During the course of the Study, the upgraded Owego Village STP was assumed to be a trickling filter plant based on current local studies. Costs presented in this report reflect a trickling filter process. However, recent design plans (by the consulting engineer for the Village) now call for an activated sludge plant to be installed at Owego Village.

Town of Union: Extension of a sanitary sewer to the Choconut Center area of the Town of Union scheduled for funding in 1976.

Town of Vestal: An interceptor sewer to the Endicott STP is scheduled for funding in 1976. This interceptor will serve the western portion of the Town of Vestal currently served by the Vestal primary STP (to be abandoned).

Town of Owego: An interceptor serving the eastern part of the town and connecting to Owego STP No. 2 is scheduled for funding in 1976.

In summary, the physical wastewater management characteristics for the Baseline Condition include:

1. Abandonment of the Vestal STP and diversion of its influent sewage via a new interceptor to the Endicott STP.

2. Upgrading of the Owego Village STP to provide secondary treatment.

3. Abandonment of the Owego Valley View STP and diversion of its influent sewage to an upgraded Owego Village STP.

4. The remaining existing STP's, including the Binghamton-Johnson City activated sludge plant, Owego STP #2 (East Owego, activated sludge plant), Owego STP #1 (West Owego, trickling filter plant), and Endicott trickling filter plant would not be expanded or upgraded.

5. The Chenango Valley area would continue on septic systems.

6. Extensions of sewage collection and treatment services would take place within the Nanticoke Creek Valley and toward Five Mile Point.

7. The sewered population to all STP's would continue to grow and sewage flows will increase.

Infiltration and Inflow

Under the Baseline Condition, no measures would be taken to correct infiltration and/or inflow problems (such as deteriorating sewers and combined sewer overflows) within the Binghamton-Johnson City and Owego Village systems.

Within the West Owego system, however, the infiltration conditions were assumed to be corrected by 1977, since the program to correct this infiltration is already well under way.

Sludge Management

Sludge management practices in the Baseline Plan were assumed to be extensions of the practices currently in operation.

1. Owego Village--Landfill disposal of digested sludge.

2. East and West Owego--Delivery of liquid sludge to farmers for application on land.

3. Endicott--Landfill disposal of digested sludge.

4. Binghamton-Johnson City--Current plans call for land application of sludge.

CHAPTER IV

METHODOLOGY FOR DEVELOPING COMPONENTS

The alternatives developed in this study were based on several components: type of wastewater treatment, degree of regionalization (number and location of treatment plants), infiltration control in the City of Binghamton sewer system, nonstructural flow reduction measures, and storm overflow controls for the Binghamton and Owego Village combined sewer systems. The array of technical components considered are outlined in Table IV-1.

As possible combinations of components were investigated, a continuing process of refinement and definition of alternatives took place, which are explained in the Plan Formulation Appendix. Some of these refinements were the result of applying logic considerations to the broad preliminary strategies, some were due to the explicit or implicit desires of the management and advisory committees, some were additions needed for a full investigation of impacts, and some were the result of technological analyses performed in more and more detail as the Study progressed. The primary intention of this Chapter is to describe the way in which technological analyses were used in the development of detailed alternatives.

TABLE IV-1

SYSTEM COMPONENTS

TYPE OF WASTEWATER TREATMENT

Secondary
Improved Secondary: Secondary plus nitrification or filtration
Land Treatment
Physical/Chemical Advanced Waste Treatment
Biological Advanced Waste Treatment

DEGREE OF REGIONALIZATION (Number and Location of Sewage Treatment Plants

1 plant (1+0)*:	Binghamton-Johnson City (two county regionalization)
1 plant (1+0):	Binghamton-Johnson City (B-JC)
2 plants (2+0):	B-JC, Endicott
3 plants (2+1):	B-JC, Endicott, East Owego
4 plants (2+2):	B-JC, Endicott, East Owego, West Owego
4 plants (3+1):	B-JC, Endicott, Chenango Valley, East Owego
5 plants (2+3):	B-JC, Endicott, East Owego, West Owego, Owego Village
5 plants (3+2):	B-JC, Endicott, Chenango Valley, East Owego, West Owego
6 plants (3+3):	B-JC, Endicott, Chenango Valley, East Owego, West Owego, Owego Village

FLOW REDUCTION MEASURES

Structural
    Infiltration Control (2, 4, or 6 mgd reduction)
Non-Structural
    Metering, water saving devices, land use zoning, pricing, public education, restrictions of sewer services, sewer ordinances

STORMWATER CONTROL OF COMBINED SEWER OVERFLOWS

Storage
Microstrainers
Dissolved air flotation
Modified biological treatment

*(1+0) refers to the number of plants in Broome County and the number in Tioga County, respectively.

TABLE IV-1 (Continued)

SLUDGE MANAGEMENT

Dewatering Processes
- Thickening, Digestion, Vacuum Filtration, Drying Beds, Lagoons

Final Disposal
- Landfill, Land Application, Incineration, Composting with Solid Waste

TIME PHASING OF SYSTEM ELEMENTS

To Meet Federal Objectives
- 1977 - Secondary Treatment
- 1983 - Best Practical Waste Treatment Technology (improved secondary)
- 1985 - Zero Discharge of Pollutants (advanced waste treatment)

To Maintain Stream Standards
- Components phased as needed to meet increased wastewater flows and water quality requirements

STRUCTURAL AND NONSTRUCTURAL FLOW REDUCTION MEASURES

The preliminary investigations of this Study had examined the implications of flow reduction, and had shown that reduced flows, particularly in the AWT strategies, could result in considerable cost savings in waste treatment. Reductions in flow could be achieved by both structural and nonstructural means. The primary examples of structural methods are sewer system renovations to reduce infiltration and inflow, whereas nonstructural methods include limitation of service area, pricing, water conservation (e.g., in the home), metering and sewer connection or building ordinances. It was recommended by ISMG that both structural and nonstructural means of flow reduction be examined further. This investigation was made in Stages II-2 and III-1.

In Stage II-2, the objective was to refine the estimates of the reduced treatment plant costs which could be gained by these flow reductions (but not to analyze the cost of implementing these measures). This was done by projecting the total flows for each service area under three categories:

1. The expected flow rates (high rates), assuming no specific reduction due either to structural or nonstructural measures. Thus, existing rates of infiltration were carried into the future, and expected increases in per capita sewage flow, continuing the trends of the past, were included.

2. Flow reductions achievable by infiltration control were estimated for the Binghamton-Johnson City system, which has been documented to have severe infiltration. In Stage II-2, a single level of flow reduction, 4.5 mgd, was used, judged on the basis of the I/I Study (Shumaker, 1974) completed during Stage II-2, to be the maximum expected cost effective level of reduction.

3. Reduced flows expected upon the application of nonstructural means. It was estimated that the maximum reduction attainable would be the continuation of the present per capita flow rates, i.e., that nonstructural means could prevent the expected future inc eases in per capita sewage flows.

Based on these projections several alternatives were designed in Stage II-2 to test the costs savings attainable by flow reduction. In order to investigate this as fully as

possible, a total of seven alternatives in Stage II-2 were examined for various levels of flow reduction, by either structural (infiltration control) and/or nonstructural measures. Alternatives with flow reduction were chosen to cover the full range of treatment levels, from secondary treatment to AWT. The flows for each service area and each level of flow reduction were projected over the planning period, and systems were then designed to meet the alternatives' objectives with that flow projection. Reduced flows result in the following means of cost savings:

1. An existing STP, or a given sized expansion of an STP, will have a longer design life, thus delaying future expansions and reducing their present worth costs.

2. Expansions of an STP are generally smaller, reducing capital costs.

3. Design capacity of an STP is smaller, reducing O&M costs.

4. The installation of specific treatment steps to meet stream DO standards, e.g., nitrification at Binghamton-Johnson City can be delayed into the future, reducing present worth of capital and O&M costs.

The conclusion of the Stage II-2 analysis was that cost savings could be realized with flow reductions. Therefore, the concept of flow reduction was carried through to Stage III-1 for further analysis.

In Stage III-1 the objective of the analysis was broadened significantly to include: flow reduction methods were to be considered in all alternatives; the analysis was to determine separately, for each alternative, the cost effective levels of flow reduction by infiltration control and nonstructural means; and specific methods of nonstructural reduction were to be analyzed and recommended.

To accomplish these objectives, the following steps were taken:

1. Information was obtained from Vernon O Shumaker, I/I consultant to Binghamton, with regard to the costs of achieving varying levels of infiltration control in that City;

2. An analysis of nonstructural means was performed (see Chapter II), which related the flow reductions attainable to the user cost of sewage service;

3. These two analyses were incorporated into a matrix analysis of system costs for alternatives at all degrees of regionalization and treatment efficiency, which enabled the identification of the cost-effective level of flow reduction associated with these system variables.

The result of the Stage III-1 analysis was that, in the final iteration, Stage III-2, each Plan included a specific level of flow reduction for infiltration control in Binghamton and, for AWT systems, nonstructural flow reduction for all service areas. The capital improvements program for each final Plan was designed on the basis of flow projections with these reductions applied.

FEDERAL AND STATE STANDARDS

The detailed design and cost work primarily considered Federal (EPA) effluent and stream standards and New York State Stream Standards. Consideration was also given to associated standards which were of importance in the Study Area context; e.g., air emission standards for incinerators, and the legal framework governing the role of institutions on wastewater management. State and Federal funding policies were also considered as discussed in Chapter II under "Per Capita Average Annual Cost," but they did not influence the engineering design and refinement of alternatives. Therefore, only the Federal effluent and stream standards and New York Stream Standards will be discussed herein. Implementation arrangements are presented in the Institutional Analysis Appendix.

FEDERAL STANDARDS

By provision of PL 92-500 and rules and regulations published by EPA pursuant thereto, the following Federal standards were applied to the design of the alternatives:

1. By 1977 all municipal STP's must accomplish secondary treatment of wastes. Secondary treatment is defined as a minimum 30 day average of 85 percent BOD and SS removal, and a maximum 30 day average of 30 mg/l, BOD and SS. In addition, secondary treatment requires a maximum effluent total coliform level of 1000/100 ml, or a fecal coliform level of 200/100 ml.

Both activated sludge and trickling filter plants were assumed to be capable of meeting the secondary treatment requirements. This minimum requirement was met by all alternatives, except the Baseline which allows secondary performance to deteriorate after 1977.

2. By 1983, all municipal STP's must accomplish Best Practicable Waste Treatment Technology (BPWTT). During Stage I and Stage II, it was estimated this would require high degrees of NOD removal at all municipal STP's. However, based on EPA preliminary guidelines, it appeared nitrification would only be required where necessary to meet stream DO standards. Alternatives which were specifically designed to accomplish nitrification at all STP's were not included in Stage III analysis, on the basis of the EPA guidance. Nitrification was only added where needed, specifically, the Binghamton-Johnson City STP.

3. By 1977, industrial effluents discharged to surface waters must achieve Best Practicable Control Technology Currently Available (BPCTCA), and, by 1983, must achieve Best Available Control Tecnology Economically Achievable (BACTEA).

4. It is the National goal that wherever attainable, an interim goal of water quality which provides for the protection and propagation of fish, shellfish, and wildlife, and provides for recreation in and on the water be achieved by 1 July 1983. Specific standards or criteria to define this "fishable-swimmable" goal have not yet been promulgated by EPA. However, it is known that the two major parameters of concern in a water's suitability for fishing and swimming are DO and coliforms. All Study Area waters are classified by New York State as suitable for fishing, and all alternatives are designed to provide a requisite DO for fish propagation. In addition, the expected coliform criteria to define suitability for swimming is a level of: fecal--200/100 ml and total--1000/100 ml, i.e., identical to the secondary treatment effluent requirements included as a minimum in all alternatives except the Baseline. Therefore for dry weather conditions, "fishable-swimmable" conditions would be met by all action alternatives.

The stormwater overflow treatment alternatives were not designed in detail to meet the expected primary contact recreation coliform standards, since the raw overflow characteristics were not specifically known. However, a literature search was conducted to estimate expected raw waste coliforms and the coliform kill to be expected from disinfection of these overflows. The expected reductions would result in stream coliform levels in the Susquehanna which are only slightly over the 1000/100 ml total coliform criterion, even though the Susquehanna below Binghamton is not classified by New York State for swimming.

5. It is the National goal that the discharge of pollutants into the navigable waters be eliminated by 1985; the elimination of pollutant discharge by 1985 is not a requirement of PL 92-500. Additionally, EPA has not yet defined "zero discharge."

However, in order to be fully responsive to PL 92-500, alternatives were formulated which would approach or accomplish this goal. As a basis of STP design, the definition of zero discharge used by the Corps of Engineers (given below) was employed. Two AWT systems were designed to accomplish this goal by 1985, one based on biological treatment (Bio AWT) and one based on physical/chemical treatment (P/C AWT). In addition to these, a land treatment alternative was included, which accomplished zero discharge during the frost-free period, but not during the remainder of the year. This was done to investigate the advantages and disadvantages of land treatment without paying the high costs associated with storing winter flows in a year-round system.

The Corps of Engineers Definition of the 1985 Goal

Because effluent limitations commensurate with the National goal of zero discharge by 1985 have not yet been established by EPA, the Corps of Engineers interpretation was used for planning purposes.

Several concepts guided the development of the Corps' definition of effluent limitations:

1. Stream assimilative capacity should not be used to provide the final degree of treatment. Elimination of the effects of point source pollution discharges and minimization of non-point source problems would eventually permit the stream to return to a non-degraded condition.

2. Recognizing that most substances are essential in certain quantities, double distillation-type treatment was ignored. Water of this type would be too pure to permit a balanced, indigenous population of fish and wildlife.

3. All water should be reserved for multiple use. The following water uses determined a list of constituents and the critical levels above which they become pollutants for: potable supply, crop irrigation, livestock watering, full body contact recreation, and freshwater fish and wildlife habitat. The most stringent level from among the five uses was chosen as the definition of the 1985 goal for each constituent.

Constituents that are harmful to the environment, even at trace levels, are presented in Table IV-2. These pollutants should be completely absent from water bodies.

TABLE IV-2

CORPS OF ENGINEERS' ZERO DISCHARGE CRITERIA
POLLUTANTS TO BE COMPLETELY ABSENT

Arsenic	Copper	Phenols
Barium	Cyanides	Selenium
Boron	Lead	Silver
Cadmium	Mercury	Zinc
	Pesticides and other synthetic organics	

In addition, the following constituents in Table IV-3 are considered a potential environmental and hygenic risk such that their absence is desirable, although presence at natural background levels may be permissable based upon an environmental assessment.

TABLE IV-3

CORPS OF ENGINEERS' ZERO DISCHARGE CRITERIA
POLLUTANTS PERMISSABLE ONLY AT NATURAL LEVELS

Antimony	Molybdenum	Tin
Beryllium	Nickel	Titanium
Cobalt	Thallium	

In the absence of determining natural background levels for a particular watercourse, the levels presented in Tables IV-4 and IV-5 are to be used in determining the maximum

TABLE IV-4
CORPS OF ENGINEERS' ZERO DISCHARGE CRITERIA
DESIGN EFFLUENT LEVELS

Constituents	Effluent Level
Total Dissolved Solids	Less than 500 mg/l in "fresh" water.
Biochemical Oxygen Demand	BOD level less than 5 mg/l. BOD level equal to or less than dissolved oxygen.
Heat	The level of which assures protection and propagation of a balanced, indigenous population of shellfish, fish, and wildlife in or on the water into which discharge is made.
Color	Less than 15 color units.
Nitrogen as Nitrate -N and Nitrite -N	Less than 4 mg/l total.
Nitrogen as Ammonia -N	Less than 0.5 mg/l.
Phosphorus as Total P	Less than 50 micrograms/liter entering a lake; or 100 micrograms/liter entering a flowing stream.
Oils and Greases	Trace
Fecal Coliform Organisms	Less than 200/100 ml.
Suspended Solids	Less than 5 mg/l.
Dissolved Oxygen	Greater than 5 mg/l.

TABLE IV-5
CORPS OF ENGINEERS ZERO DISCHARGE CRITERIA
ADDITIONAL DESIGN PARAMETERS

Constituent	Effluent Level
Virus	Inactivated but present at trace levels.
Surfactants	Trace
Fecal Streptococci	Inactivated but present at trace levels.
Tastes and Odors	None offensive.
Flotables	None
Settleable Solids	Trace
Volatile Solids	Trace
Gamma Radiation	Trace
Alpha Radiation	Less than one pico curia/liter.
Beta Radiation	Less than 100 pico curie/liter.
Turbidity	Less than five Jackson units.
Chemical Oxygen Demand	Less than 10 mg/1.
pH	Between 6.0 and 8.5
Alkalinity	Less than 100 to 130 mg/1 when pH is between 6.0 and 7.0
Carbon Dioxide	Less than 25 mg/1.
Sulfates	Less than 100 mg/1.
Calcium	Less than 30 mg/1.
Chlorides	Less than 250 mg/1.
Sodium	Less than 10 mg/1.
Magnesium	Less than 125 mg/1.
Fluorides	Varies from 1.7 mg/1 at $10^{o}C$ to 0.8 mg/1 at $30^{o}C$.
Aluminum	Less than 1 mg/1.
Bicarbonates	Less than $\pm$ 50 mg/1 variation over ambient concentrations
Manganese	Less than 0.05 mg/1.

acceptable levels for design. These effluent levels may be altered on the basis of an environmental assessment of their impacts.

The specified levels are upper bounds and not average values. The constituents in Tables VI-2, VI-3, and VI-4 comprise the minimum acceptable group that must be considered in systems design. Other constituents (see Table IV-5) should be considered as appropriate, depending upon characteristics of the region. The advanced waste treatment systems designed for the Binghamton Wastewater Management Study would meet or approach most of these criteria for the National goal.

NEW YORK STATE STREAM STANDARDS

The Susquehanna River and its tributaries in Broome and Tioga Counties have five basic water quality classifications (Table IV-6 and Figure IV-1). The main stem of the Susquehanna has A, B, and C classifications. The Chenango and Tioughnioga Rivers and Cayuta Creek are Class B waters. The main stems of major creeks such as Owego and Nanticoke are Class C trout waters and the remaining small streams, many of which are intermittent, have a D classification.

The waters impacted upon by the alternatives (Susquehanna and Chenango Rivers) are classified as either B or C, non-trout. Table IV-7 summarizes the standards for these classifications which were pertinent to design of wastewater management alternatives.

APPLICATION OF STANDARDS TO ALTERNATIVES

Coliform

All alternatives with the exception of the Baseline were designed to meet, at the minimum, secondary treatment as defined by EPA, and to provide Class B water quality in the Susquehanna and Chenango Rivers. The total coliform requirement for secondary treatment is more stringent than the New York State swimming (Class B) water standard. All

TABLE IV-6

NEW YORK STATE CLASSIFICATION OF SURFACE WATERS

Classification	Best Usage
A	Source of water supply for drinking, culinary or food processing purposes, and any other usages.
B	Primary contact recreation and any other use except as a source of water supply for drinking, culinary, or food processing purposes.
C	Suitable for fishing and all other uses except as a source of water supply for drinking, culinary, or food processing purposes, and primary contact recreation.
D	These waters are suitable for secondary contact recreation, but due to such natural conditions as intermittency of flow, water conditions not conducive to propagation of game fishery or stream bed conditions, the waters will not support the propagation of fish.

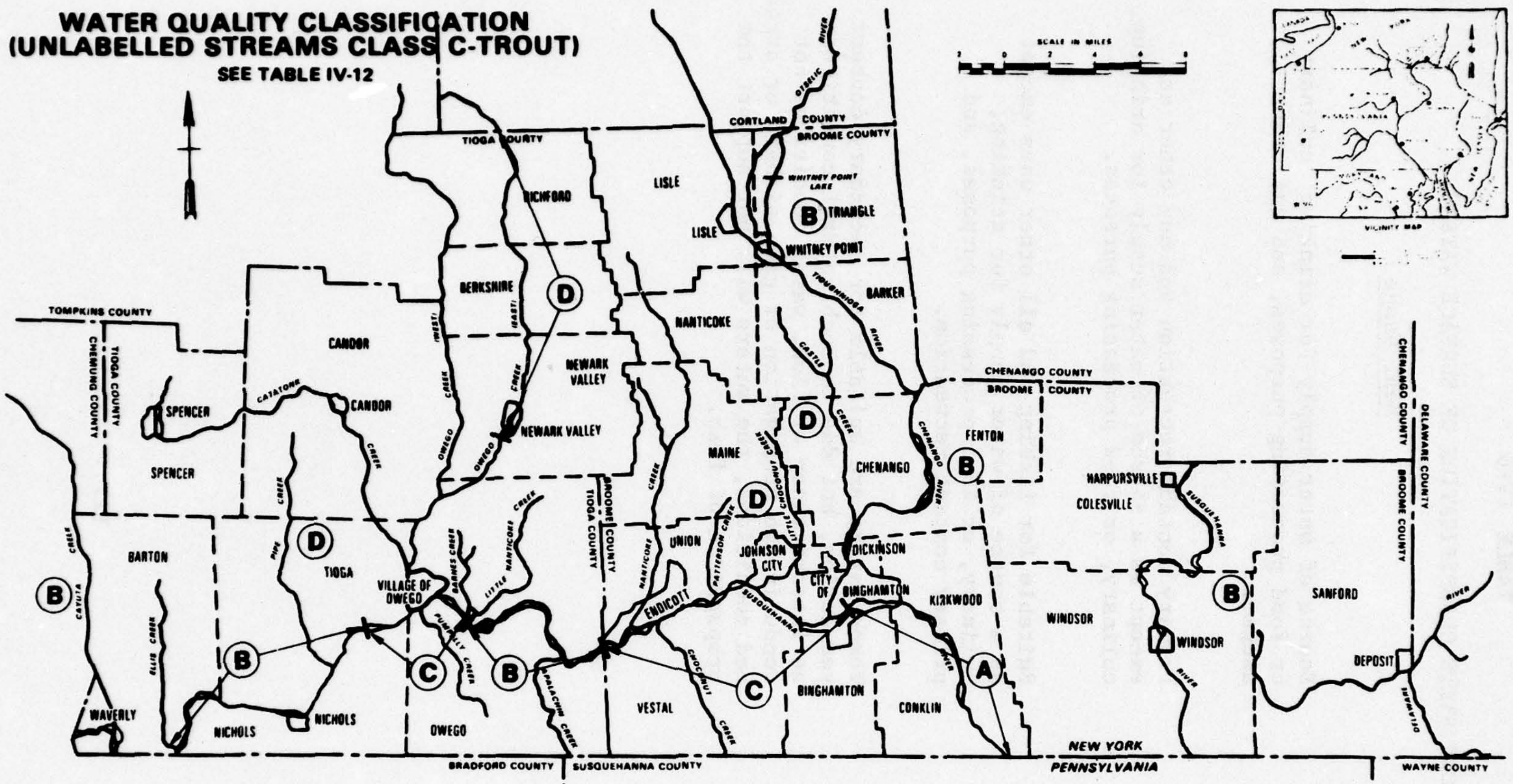

WATER QUALITY CLASSIFICATION
(UNLABELLED STREAMS CLASS C-TROUT)
SEE TABLE IV-12
SCALE IN MILES
VICINITY MAP
TOMPKINS COUNTY
TIOGA COUNTY
CHEMUNG COUNTY
CORTLAND COUNTY
BROOME COUNTY
CHENANGO COUNTY
DELAWARE COUNTY
BRADFORD COUNTY
SUSQUEHANNA COUNTY
WAYNE COUNTY
NEW YORK
PENNSYLVANIA
SPENCER
CANDOR
RICHFORD
BERKSHIRE
NEWARK VALLEY
LISLE
TRIANGLE
WHITNEY POINT
WHITNEY POINT LAKE
BARKER
NANTICOKE
MAINE
CHENANGO
FENTON
COLESVILLE
HARPURSVILLE
SANFORD
DEPOSIT
WINDSOR
KIRKWOOD
CONKLIN
BINGHAMTON
CITY OF BINGHAMTON
DICKINSON
JOHNSON CITY
UNION
ENDICOTT
VESTAL
TIOGA
VILLAGE OF OWEGO
OWEGO
NICHOLS
BARTON
WAVERLY
CATATONK
CREEK
OWEGO CREEK
CASTLE CREEK
TIOUGHNIOGA RIVER
OTSELIC RIVER
CHENANGO RIVER
SUSQUEHANNA RIVER
DELAWARE RIVER
NANTICOKE CREEK
LITTLE NANTICOKE CREEK
PATTERSON CREEK
LITTLE CHOCONUT CREEK
CHOCONUT CREEK
APALACHIN CREEK
PUMPELLY CREEK
BARNES CREEK
PIPE CREEK
CAYUTA CREEK
A
B
C
D

FIGURE IV - 1

TABLE IV-7

NEW YORK STATE STREAM STANDARDS

Classification	Best Usage	DO Minimum Daily Average	DO Minimum Instantaneous	Maximum Coliform Total Median Per 100 ml	Maximum Coliform Fecal Median Per 100 ml	Maximum NH_3 mg/1 as NH_3
B	Primary Contact Recreation	5.0	4.0	2,400	200	2.0
C	Fishing	5.0	4.0	10,000	2,000	2.0

alternatives would meet the New York State coliform standard in dry weather and design storm (1.25 inches in 24 hours) conditions. During design storm conditions, on-site treatment provided to overflows would just barely violate the EPA total coliform level of 1000/100 ml, and this only in waters not classified for swimming.

Ammonia

The instream ammonia standard could be met by providing requisite levels of nitrification in the STP's. The allowable load was computed at design dry weather low flow, i.e., the minimum average seven consecutive day flow expected at a one in ten year return period (MA7CD/10). Because of the existing treatment efficiencies and the particular assimilation dynamics of the Susquehanna River, secondary treatment alone at all plants would be sufficient to meet this standard, as determined by the model from BOD-DO relationships (discussed below). Therefore, in the design of the alternatives the ammonia standard was not specifically considered.

Dissolved Oxygen

The NYSDEC dissolved oxygen standards of 4.0 mg/l minimum instantaneous and 5.0 mg/l minimum daily average, on the other hand, were central to the design of the alternatives for two basic reasons:

1. Because of the assimilation dynamics of the River, these standards would be violated under the Baseline Plan, and, therefore, designs were generated specifically to meet them; and,

2. Until the final iteration, there was a great deal of uncertainty regarding the way in which the standards were to be used in the planning effort.

The uncertainty with regard to the use of the DO standards arises from the fact that the standards are primarily used for enforcement purposes, i.e., they provide a numerical standard against which the results of a monitoring program can be compared. For these purposes there is no uncertainty since the results of a 24-hour monitoring program, with samples every two to four hours, can be averaged to check compliance with the 5.0 mg/l standard, and the minimum single value can be compared to the 4.0 mg/l minimum instantaneous allowable.

For planning purposes, however, reliance on predicting the DO resulting from any alternative was placed on a mathematical model, in this case one developed by Lawler, Matusky and Skelly Engineers, for NYSDEC as part of NYSDEC's 303 Basin Planning Program. There are limits to which such a model can precisely predict the actual DO profiles in a river under critical conditions such as loading, river flow, and temperature.

In the real case, at any point downstream from a discharge of BOD and with constant (steady-state) conditions of river flow and temperature, river DO will fluctuate over the day as a result of two causes:

1. The effluent BOD from an STP discharge will fluctuate, due to fluctuations of flow and BOD concentration in the raw waste (these variations are fairly small); and,

2. The photosynthetic nature of algae (which exist in the Susquehanna) which causes increases in DO during the day and reductions during the night.

These fluctuations, particularly those due to algae, cannot be predicted without a sophisticated dynamic water quality model, and, more importantly, would require extremely extensive sampling programs to provide a basis for model calibration since algae activity is a function of numerous parameters (such as season of the year, temperature, light intensity, river velocity, depth, turbidity, species dominance). NYSDEC, in order to avoid the problems and cost associated with predictions of DO fluctuations over the day, required a different approach in their 303 models: the model was to predict what the response of the river would be if there were no algae. This greatly simplifies the procedure since only a single sampling program is required, the results of which can be analyzed to determine what the DO profile would have been, on the sampling day, if there had been no algae present. These results can then be used as a basis for model calibration and verification, which provides evaluation of all factors involved in the DO response, with the exception of the algae effect. Predictions can be made of DO profiles under critical conditions of river flow and temperature for any combination of waste loads.

The problem which arises from this simplification is that, since there are algae in the River, the model does not predict either the minimum daily average or the minimum instantaneous DO rather, it predicts a hypothetical DO, the relationship of which to the standards is not known. The use of such a profile or model in the planning effort was the

uncertainty which existed, that is, what predicted hypothetical DO value, 4.0 mg/l or 5.0 mg/l, would be accepted as meeting the New York State Standards? NYSDEC in its 303 Basin Planning (waste load allocation) Program, had been using 4.0 mg/l predicted DO as the required value; EPA Region II, on the other hand, had some objection to this and opted for the more conservative approach.

This problem was discussed at several CAC, TAC, and ISMG meetings. The outcome of these discussions was an agreement that no alternative should be rejected merely because it did not meet a hypothetical minimum DO of 5.0 mg/l, but that no alternative (except Baseline) should have a predicted DO of less than 4.0 mg/l. Therefore, this study carried through "4.0 mg/l" plans, as well as "5.0 mg/l" plans, with an assessment made of the differences between the two in terms of impact on stream biota.

The model used in this Study was a simplified version of the one used under contract to NYSDEC, based on the observation that neither non-point sources or the STP's downstream of Endicott, i.e., the three Owego Plants, have a measurable effect on river DO, even at critical conditions. Test runs of both models were made to verify the reproducibility of the complete model's results by the simplified version.

Use of the model was as follows:

1. For alternatives which had a specific treatment objective (e.g., baseline, secondary treatment, AWT, and the land application alternatives) the model was used merely to compute the DO resulting from the application of the treatment constraints. These model runs used year 2020 discharge loadings and critical stream conditions, i.e., summer temperatures and MA7CD-10 flow.

2. For those alternatives which had a specific water quality objective (e.g., 4.0 or 5.0 mg/l), the model was used on a trial and error basis to determine in what year the standard would be violated by the application of secondary treatment at all plants. This became the design year for installation of the next higher degree of treatment for at least one STP. With the higher degree of treatment installed, the model was again used on a trial and error basis to determine if, at any time in the planning period, further treatment was necessary to meet the target DO level.

The latter procedure was greatly simplified, in that secondary treatment alone was sufficient to meet 4.0 mg/l throughout the planning period, and additional treatment to

meet 5.0 mg/l consisted, in all cases of wastewater flow, of nitrification at the Binghamton-Johnson City STP only. The year in which nitrification at the Binghamton-Johnson City STP was necessary varied since the performance of secondary treatment at Binghamton-Johnson City is a function of both influent flow and BOD. Thus, application of flow reduction methods would delay the need for nitrification, and inclusion of new service areas, such as Chenango Valley, may require nitrification sooner than otherwise (see Chapter V).

There is a final point to be mentioned with regard to the DO standards and the model used for design purposes. During Stage II-2, the model was finalized under the contract with NYSDEC. This finalization included a reevaluation of the reaeration coefficient. This reevaluation was based on cross sectional geometry and flow information received after the completion of Stage II-1, and resulted in a significant increase in the estimated reaeration coefficient at low flow conditions. As a result, whereas Stage II-1 had designed nitrification for both the Binghamton-Johnson City and Endicott STP's to meet 5.0 mg/l, subsequent iterations designed nitrification only at Binghamton-Johnson City.

COST-EFFECTIVENESS

Cost-effectiveness can be defined as producing a given objective or goal at minimum cost. The present worth analysis was primarily used in comparing different alternatives which had the same overall objectives, e.g., comparing two schemes designed to meet the 5.0 mg/l DO objective but with different degrees of regionalization. However, the cost-effective analysis was also considered in the actual design in setting design lives for component parts of an alternative.

The methodology used to assign design lives for different types of wastewater control units was discussed in detail in Chapter II, and showed that it is economical to design structures with design lives much shorter than the commonly accepted 20 to 30 years for sewage treatment plants. Factors such as growth rate, interest rate, economy of scale, and

the relative dependence of O&M costs on flow or design capacity, all influence the theoretically optimal design life from an economic, or cost-effectiveness, point of view. For the Study Area situation, STP design lives as small as eight years are economical in the high growth area of Chenango Valley.

However, in the practical case, there were several factors (with respect to fund raising, planning, and construction) which indicated design lives larger than the theoretical optimum should be used.

MINIMIZE ENVIRONMENTAL IMPACT

The environmental impact assessment analyzed the alternatives in each iteration and was used to compare alternatives on a basis other than cost-effectiveness. The iterative nature of the planning process (i.e., formulation, impact assessment, and then evaluation) assured the integration of the environmental impact assessment in that alternatives were accepted or rejected for further refinement based at least partly on the impact assessment. Notwithstanding this overall integration of impact assessment in plan refinement, there were design decisions made within an iteration with the intent of minimizing environmental impact.

A few examples of such decisions are:

1. Interceptor Route Selection. Interceptor routes were selected in the field by coordination between a design engineer and an environmental assessor. Within reasonable limits of economics, interceptor routes were selected to avoid such sensitive areas as commercial areas, roads, stream corridors, wetlands, and mature forests. Proposed interceptor routes for each of the final plans are shown in Plates 2, 3, 4, and 5.

2. Land Treatment Sites. A serious consideration with regard to large land treatment sites was whether to invoke eminent domain on existing residents. If all existing residents were moved by eminent domain, then design, construction, and operation of the system becomes simplified,

in that very large contiguous blocks of land could be devoted to the land application system. However, invoking eminent domain would create a socio-economic impact. It was judged that the latter impact would be more adverse than the benefits of parceling the land for land application. Therefore, the system was designed, by use of liberal buffer zones, to be as compatible as possible with existing land uses and avoiding as many residents as possible.

3. Location of Chenango Valley STP. Initially, there were three possible sites for the location of this plant developed by R. J. Martin, Consulting Engineer, for an analysis of whether to treat the wastes from Chenango Valley separately from the Binghamton-Johnson City wastes. Based on environmental considerations, primarily with regard to compatability with adjacent land uses, only one of these sites was considered suitable, and was used as the site in the design of a separate Chenango Valley STP. The site is the same one which was the final choice of R. J. Martin (1975).

RECEIVING WATER QUALITY (DO) MODEL

GENERAL

Estimates of the dissolved oxygen level in the Susquehanna River resulting from each of the proposed treatment systems were made with the use of a mathematical model developed by Lawler, Matusky and Skelly Engineers as part of NYSDEC's 303 Basin Planning Program. The model used in this analysis is the classical Streeter and Phelps equation. This equation has proven to be accurately descriptive of the response to organic wastes of streams such as the Susquehanna, and is readily adaptable to solution by computer. The basis of the equation is the assumption that the rate of deoxygenation, or depression of dissolved oxygen by biological activity, is proportional to the amount of organic material present, and the rate of reoxygenation (taking into solution of atmospheric oxygen), is directly proportional to the existing oxygen deficit (amount below oxygen saturation).

The equation is:

$$\frac{dD}{dt} = k_d\ L - k_2\ D + k_n\ \ NOD$$

Where: D = dissolved oxygen deficit, mg/l

t = time of travel, days

L = river BOD, mg/l

NOD = river NOD, mg/l

k_d = deoxygenation coefficient of BOD, per day

k_2 = reoxygenation coefficient, per day

k_n = deoxygenation coefficient of nitrogenous oxygen demand, per day

The integrated form of the equation is:

$$D_t = \frac{k_d L_0}{k_2 - k_r}\ [e^{-k_r t} - e^{-k_2 t}] + D\ e^{-k_2 t} + \frac{K_n L_n}{k_2 - k_n}\ [e^{-k_n t} - e^{-k_2 t}]$$

where: K_r = BOD removal coefficient, per day

L_0 = river ultimate BOD at t = 0, mg/l

D_0 = river dissolved oxygen deficit at t = 0, mg/l

L_n = river ultimate nitrogenous demand at t = 0, mg/l

k_n = nitrogenous deoxygenation coefficient, per day

In the solution of this equation, the assumption is made that over time, t, steady-state conditions prevail, i.e., that conditions of flow, temperature, waste loads and initial dissolved oxygen and BOD do not vary with time. This assumption is accounted for in the modelling by segmenting the river reaches appropriately at points of change of conditions, e.g., input waste loads, dams, and tributaries.

The computerized solution of the model used in this study is a generalized one, in that it includes oxygen depletion by organic bottom deposits, as well as the factors specified in the above equation. The field surveys showed that there are no significant bottom deposits in any of the reaches studied,

due to a combination of sewage treatment and relatively high velocities in those rivers. Therefore, the effect of bottom deposits on DO was not considered in the analysis. The computer program does, however, provide the capability of accounting for these effects if they are present.

MODIFICATION NECESSITATED BY STEADY-STATE MODEL

Although a steady-state Streeter-Phelps equation is normally used for water quality (DO) modelling, the actual variation of dissolved oxygen in a river is an unsteady phenomenon. Two factors, time-varying waste loads and plant photosynthesis, account for most of the dynamic nature of river DO. In order to apply the equation, both of these variables must either be added to the DO model, or alternatively, removed from the sampling data used for model validation. Since neither source can be predicted analytically with much accuracy, the use of empirical techniques to remove the effects from sampling data was chosen in the study for NYSDEC, on which the present analysis is based. Those techniques are discussed below.

Slug Sampling

The time-variable effects of waste loading are nullified by observing the effects of a wastewater "slug" as it moves down the river.

A two-hour composite sample of wastewater was taken at a time when waste loadings are typically high (e.g., late morning). Downstream water quality sampling times were based on river travel times (measured from the time of wastewater sampling) so that all water quality sampling is timed to correspond to the passage of the two hour wastewater "slug." Downstream waste discharges and tributary inflows are also sampled to bracket the time the slug passes them. In addition to this two hour water quality sampling, DO was measured over a 16 to 24 hour period bracketing the slug, to document diurnal DO fluctuations.

Times of travel can be determined either from dye studies or taken directly from USGS travel time versus discharge graphs. Dye studies were not generally successful during

the stream surveys due to equipment malfunctioning, so that sampling was based on USGS travel times. Some measurements were possible, however, and close agreement with USGS times was obtained.

"Scrubbing" Photosynthesis and Respiration Effects from Survey DO Data

The ultimate purpose of the study for NYSDEC was to provide a basis for allocating waste loads on each river reach to assure that applicable DO standards are not violated at critical low stream flow conditions. In this allocation, the effect of oxygen production or depletion by plant life was to be considered zero, i.e., no "credit" would be allowed to a waste discharge for a net production of oxygen which might occur; and no "debit" would be charged to the discharge for a net decrease in oxygen, which also might occur. Therefore, the effects of photosynthesis and respiration were not to be accounted for in the production model runs, i.e., in the assessment of the effects of existing waste loads at critical low flow conditions, and in subsequent waste load allocation runs.

This was accomplished by a rectification of the field data (diurnal DO) to determine what the measured DO profile would have been if no plant life had been affecting river DO during the field survey. Verification of the various parameters affecting the DO profile was based on this theoretical profile, and these parameters, modified as necessary, were used in the production runs. The specific technique used to accomplish this rectification is referred to in this report as "scrubbing."

Photosynthesis is a basic and complex process which plays a role in many streams. The process also makes the stream DO system more complex than the stream without photosynthesis. Whether the plant production of oxygen should be taken as an oxygen source has been argued for more than a decade, although the pioneers of stream DO models, Streeter and Phelps, and other workers in limnology, noticed its existence and tried to analyze its effect on the stream DO.

If suspended algae were the only plant in the stream, the effect could be assessed by algal concentration or other methods. However, other forms of plants, such as attached algal or rooted plants, make it extremely difficult to

measure the effect, even by in-situ study. In order to resolve this difficulty and obtain a reasonable measurement of plant effect on the stream DO, a procedure was developed during the NYSDEC study, which would remove the effects of photosynthesis and respiration from survey DO, simply using the survey stream diurnal DO data and other system parameters. Because of the unique nature of this procedure, it is described in detail herein.

The procedure starts from the deterministic model which describes the real system of stream DO (with some simplification):

$$\frac{\partial c}{\partial t} = \frac{\partial}{\partial X}\frac{QC}{A} - k_dL - K_nL_n + K_2D + (P-R) \tag{1}$$

C = DO Concentration (mg/l)

Q = Stream flow (cfs)

A = Stream cross sectional area (sq. ft.)

t = time of travel (day)

X = distance downstream (ft)

D = DO deficit (mg/l)

K_d = Deoxygenation rate coefficient (l/day)

L = stream ultimate BOD (mg/l)

K_n = Nitrification rate coefficient (l/day)

L_n = Nitrogenous oxygen demand (mg/l)

K_2 = reaeration coefficient (l/day)

P = Plant and algae photosynthesis rate (mg/l-day)

R = Plant and algae respiration rate (mg/l-day)

For constant flow and with proper segmentation of a stream, equation (1) can be simplified to:

$$\frac{c}{t} = -V\frac{c}{x} - K_dL - K_nL_n + K_2D + P\text{-}R \qquad (2)$$

Where $V = \frac{Q}{A}$, stream velocity (ft/day).

Isolating (P-R) from equation (2):

$$(P\text{-}R) = \frac{c}{t} + V\frac{c}{x} + K_dL + K_nL_n - K_2D \qquad (3)$$

Those terms on the right side of equation (3) can be obtained from stream samples analyzed for diurnal DO, temperature and long term BOD; K can be calculated from the following equation proposed by Tsivoglou et al:

$$K(2) = C\frac{h}{t} \qquad (4)$$

Where h = water surface elevation change (ft)

t = time of flow corresponding to h, (day)

C = escape coefficient [ft(-1)]

For a first approximation, equation (3) can be simplified as:

$$P\text{-}R = \frac{dt}{dt} - K_2D \qquad (5)$$

Here, it is assumed that the terms neglected from equation (3) are small compared to the remaining terms. In general, this is the case found in the sampling data which showed that plants contribute more than 85 percent of the unsteady state variation in DO data. Where other factors, such ad K(d)L, were not insignificant, the proper term was inserted.

The new photosynthesis rate (P-R) was computed as a function of time for each diurnal sampling station as:

$$P\text{-}R = \frac{\Delta c}{\Delta t} - K_2D \qquad (6)$$

Where Δt = short time increment, typically 2 hours

Δc = change in DO during t (mg/l)

A smoothed DO curve (over the day) was used in this computation. DO deficit (D) used was the mean value for the two-hour period. Generally, the computed rates, (P-R), were positive in daytime and negative in night time, as affected by light, plant population, water quality, and other environmental conditions.

In the next step, the effective photosynthesis and respiration rate (P-R), for the time the "slug" (of waste) passed a sampling station was determined for each consecutive sampling station as:

$$\overline{P-R} = W_U \; \overline{(P-R)_U} + W_D \; \overline{(P-R)_D} \qquad (7)$$

The relative weightings Wu (upstream station) and Wd (downstream station) were quantified on the assumption that the downstream station is more representative of the two stations, since the resulting P-R will be used for scrubbing the plant effect from the survey result for the downstream station. The values of Wu and Wd were assigned 0.4 and 0.6, respectively, which can be assessed through the use of K(2) and time of travel between the two stations. It is unnecessary that Wu and Wd be some fixed value; rather they can vary around the values given above. With similar reasoning, (P-R)u and (P-R)d were assessed by using a weighted average, instead of arithmetic average, at each station.

Once photosynthesis and respiration rates were calculated, the change in DO caused by plants was obtained by direct integration of equation (5), or its equivalent, equation (6) as:

$$DO = \frac{\overline{P-R}}{K_2} \left(1 - e^{-K_2 T}\right) \qquad (8)$$

Where T = time of travel between upstream station and downstream station.

Let Cu = DO at upstream station at time when slug passes.
Cd = DO at downstream station at time when slug passes.

The slug DO concentration which would have been observed at downstream station in the absence of plant and algae growth was thus estimated as

$$C_D{}^1 = C_D - \frac{\overline{P-R}}{K_2} \left(1 - e^{-K_2 T}\right) \qquad (9)$$

This procedure was repeated for each segment moving downstream, with different correction factors for each segment, depending on computed (P-R) rates and time of day.

This above procedure has been applied to several streams with fairly high algae and plant growth in south central New York State. One of the streams, Tioughnioga River, will be discussed for demonstration of the scrubbing process.

The Tioughnioga River is a small stream with MA-7-CD-10 flow of 35 cfs at Cortland, the major waste source on the stream. During the survey period, the flow at Cortland was 55 cfs, average depth about 1.1 feet and velocity 11.2 miles per day. The River has high algae and plant growth, with a daily average primary production of more than 11 mg/l in some sections and daily DO fluctuation from 4.3 to 15 mg/l as the maximum and the minimum. Figure IV-2 shows the locations of sampling stations, surveyed maximum and minimum DO, slug DO and others. Figure IV-3 gives the first two stations' diurnal DO which was used with other pertinent parameters to compute the photosynthesis and respiration rates at the same stations shown on Figure IV-4, by using equation (6). Knowing travel time along the stream, the slug DO readings were traced downstream and plotted as on Figure IV-2. The time of day at each station was used to find the effective photosynthesis and respiration rate for each two consecutive stations and applied to equation (7). As the effect of primary production will not cease at the end of the segment, the correction factor (DO change due to algae and plant activity) was carried down to downstream segments, and decreasing along the stream due to diffusing out through K(2) activity, with results similar to Doe(-K(2)t). Subtracting the correction factor from slug DO, the scrubbed DO was obtained and the procedure was repeated downstream until all the effective survey stations were used and the final results were plotted as scrubbed DO on Figure IV-2.

APPLICATION OF THE MODEL FOR THE BINGHAMTON WASTEWATER MANAGEMENT STUDY

The computer program used in this study was a simplified version of the one used in the NYSDEC study ("Wastewater Assimilative Capacity Study for the Susquehanna River Basin"). The simplifications consisted of the elimination of

Figure IV-2

Tioughnioga River:
D.O. Along the Stream
(Slug, Scrubbed and Surveyed,
Maximum and Minimun)

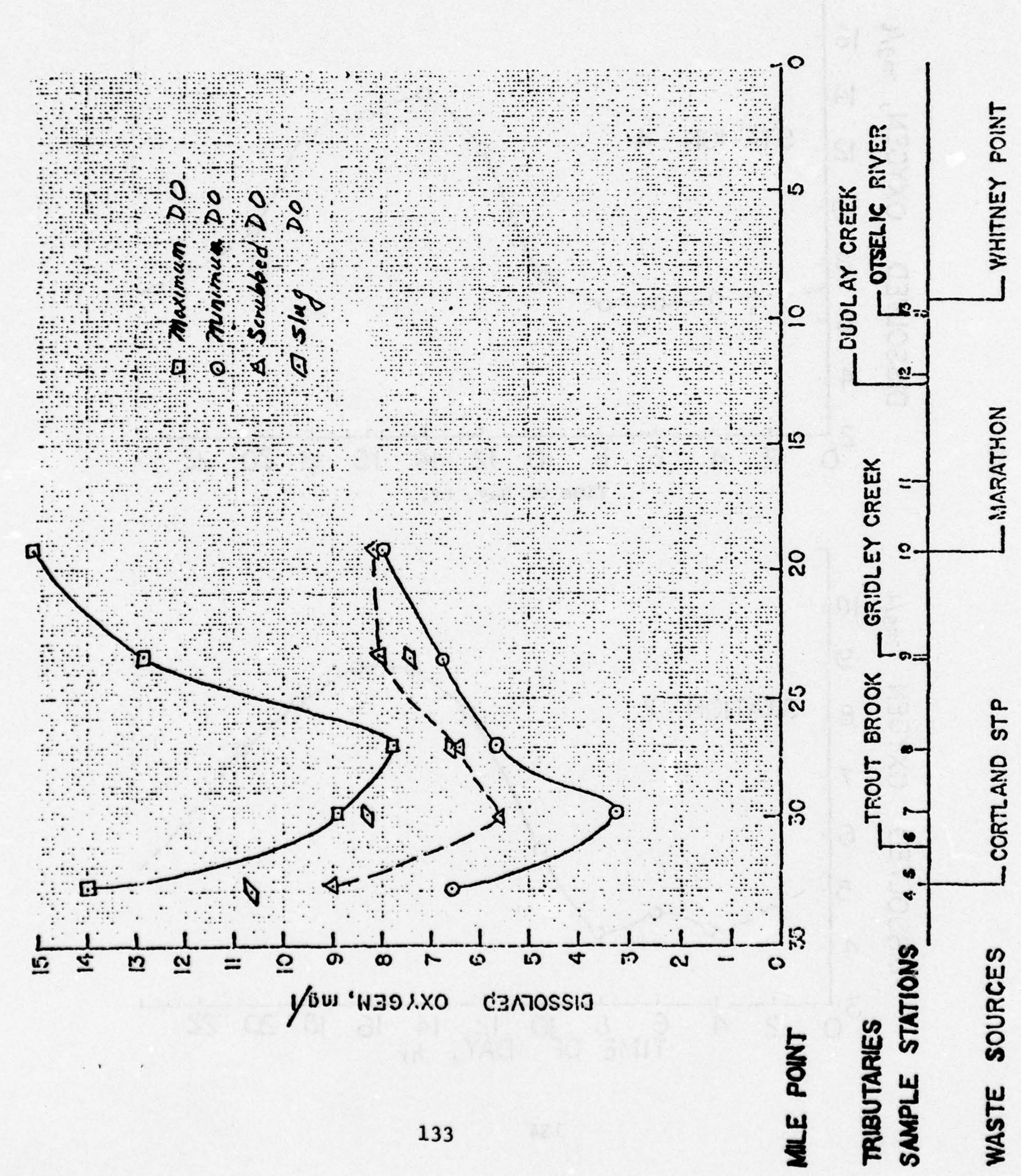


Figure IV-3

Tioughnioga River

DIURNAL DISSOLVED OXYGEN

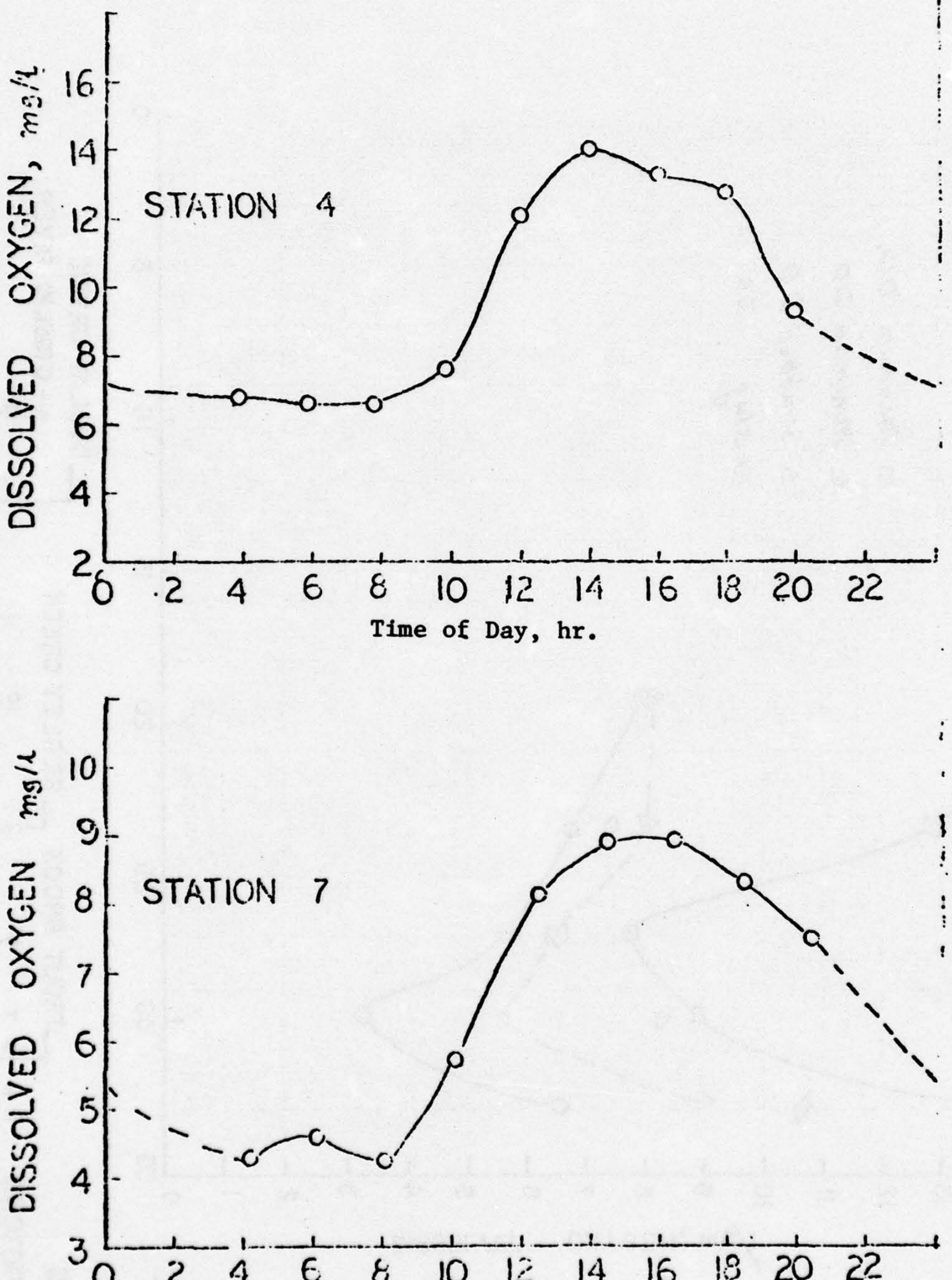
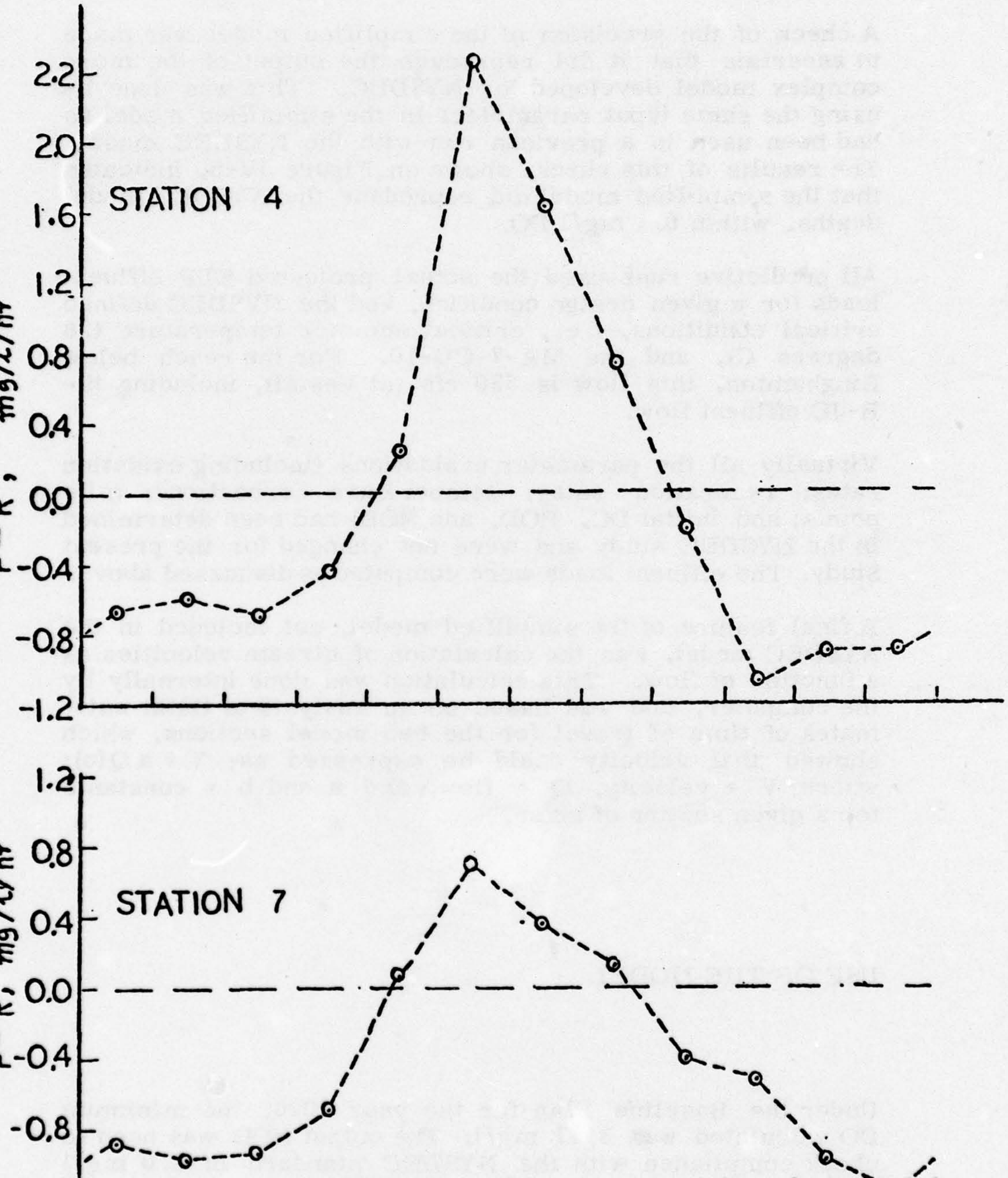


Figure IV-4
TIOUGHNIOGA RIVER
DIURNAL VARIATION OF (P-R)



those factors in the original model found to have a negligible effect on river DO, including the Owego treatment plant loads, distributed waste loads, loss of BOD by settling, and dam reaeration.

A check of the precision of the simplified model was made to ascertain that it did reproduce the output of the more complex model developed for NYSDEC. This was done by using the same input parameters in the simplified model as had been used in a previous run with the NYSDEC model. The results of this check, shown on Figure IV-5, indicated that the simplified model did reproduce the NYSDEC model depths, within 0.1 mg/l DO.

All predictive runs used the actual projected STP effluent loads for a given design condition, and the NYSDEC defined critical conditions, i.e., critical summer temperature (26 degrees C), and the MA-7-CD-10. For the reach below Binghamton, this flow is 330 cfs (at Vestal), including the B-JC effluent flow.

Virtually all the parameter evaluations (including oxidation rates; reaeration rates; temperature corrections; mile points; and initial DO, BOD, and NOD) had been determined in the NYSDEC study and were not changed for the present Study. The effluent loads were computed as discussed above.

A final feature of the simplified model, not included in the NYSDEC model, was the calculation of stream velocities as a function of flow. This calculation was done internally by the computer, and was based on an analysis of USGS estimates of time of travel for the two model sections, which showed that velocity could be expressed as: V = a Q(b); where: V = velocity, Q = flow, and a and b = constants for a given section of river.

USE OF THE MODEL

Under the Baseline Plan for the year 2020, the minimum DO calculated was 3.47 mg/l. The output NOD was used to check compliance with the NYSDEC standard of 2.0 mg/l NH3 maximum from the following relationship:

COMPARISON OF SIMPLIFIED MODEL TO NYSDEC MODEL

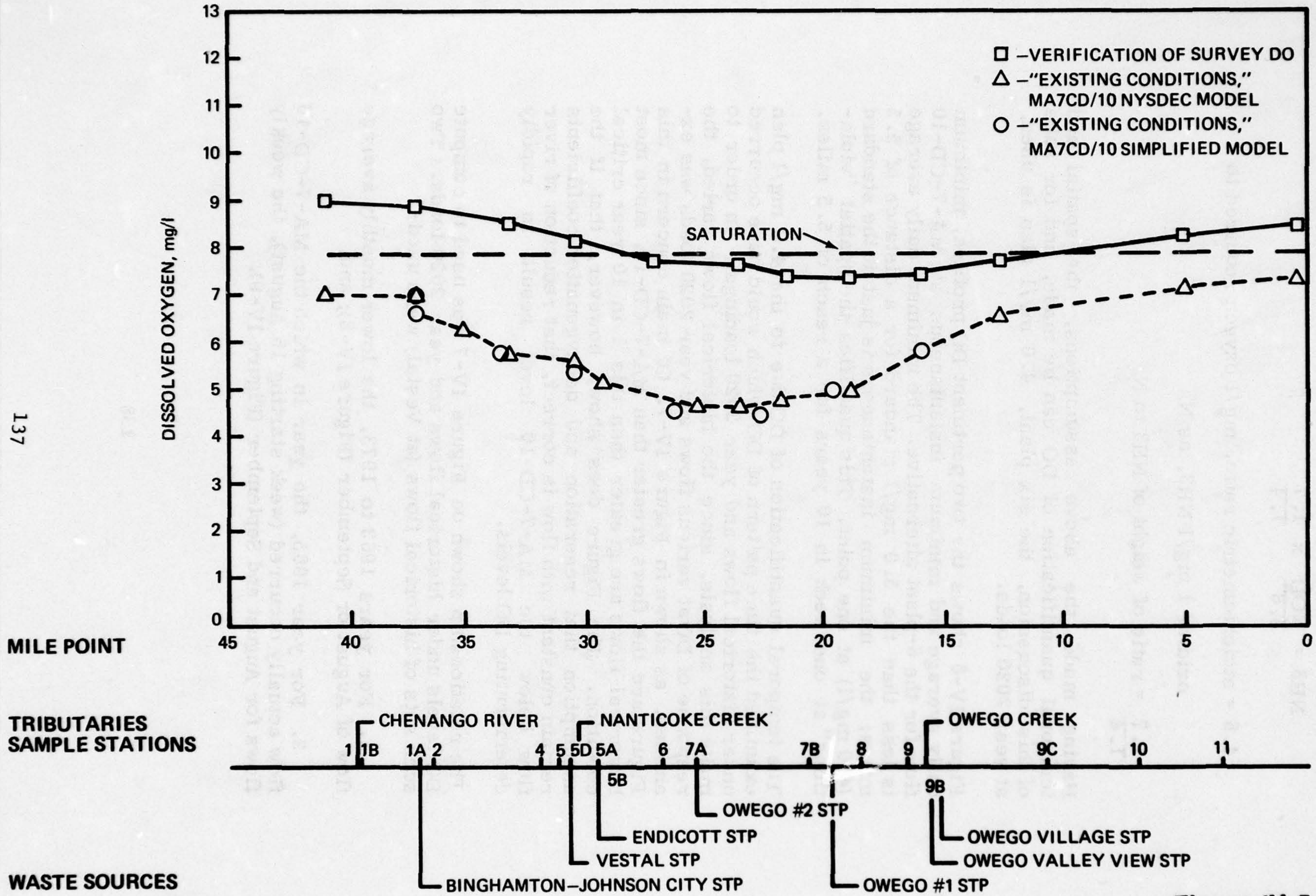


Figure IV-5

$$NH3 = \frac{NOD}{4.6} \times \frac{1.7}{1.4}$$

4.6 = stoichiometric ratio, mg/l oxygen required to oxidize 1 mg/l NH3, as N

$\frac{1.7}{1.4}$ = ratio of weight of NH3 to N

Having made the above assumptions, the spatial and temporal quantification of DO can be made, and for sake of this discussion, the six plant, 4.0 mg/l plan is used, at year 2020 loads.

Figure IV-6 shows the two pertinent DO profiles, minimum daily average and minimum instantaneous at MA-7-CD-10 flow for the 6-plant alternative. The minimum daily average is less than the 5.0 mg/l standard for a distance of 5.5 miles; the minimum instantaneous is just at the standard (4.0 mg/l) at one point. This quantifies the spatial "violations" at one week in 10 years for a reach of 5.5 miles.

The temporal quantification of DO due to the 4.0 mg/l plan examined the time pattern of DO which would have occurred under historical flows and year 2020 loadings. In order to make this analysis, since the historical flows varied, the response of DO at various flows and year 2020 loads was examined, as shown in Figure IV-7. Of main concern in this Figure are the flows greater than MA-7-CD-10, since most historical flows are greater than this 1 in 10 year critical condition. The Figure does show, however, that if the assumption that reaeration and deoxygenation coefficients remain constant with flow is correct, that reduction of river flow below the MA-7-CD-10 level result in rapidly deteriorating DO levels.

The relationship shown on Figure IV-7 was used to compute DO levels under historical flows and year 2020 loads. Two such sets of historical flows (at Vestal) were used:

1. For years 1963 to 1973, the lower monthly average flow of August or September (Figure IV-8); and,

2. For year 1965, the year in which the MA-7-CD-10 flow actually occurred (week starting 15 August), the weekly flows for August and September (Figure IV-9).

EFFECT ON SUSQUEHANNA RIVER DO: 6 PLANT SECONDARY TREATMENT

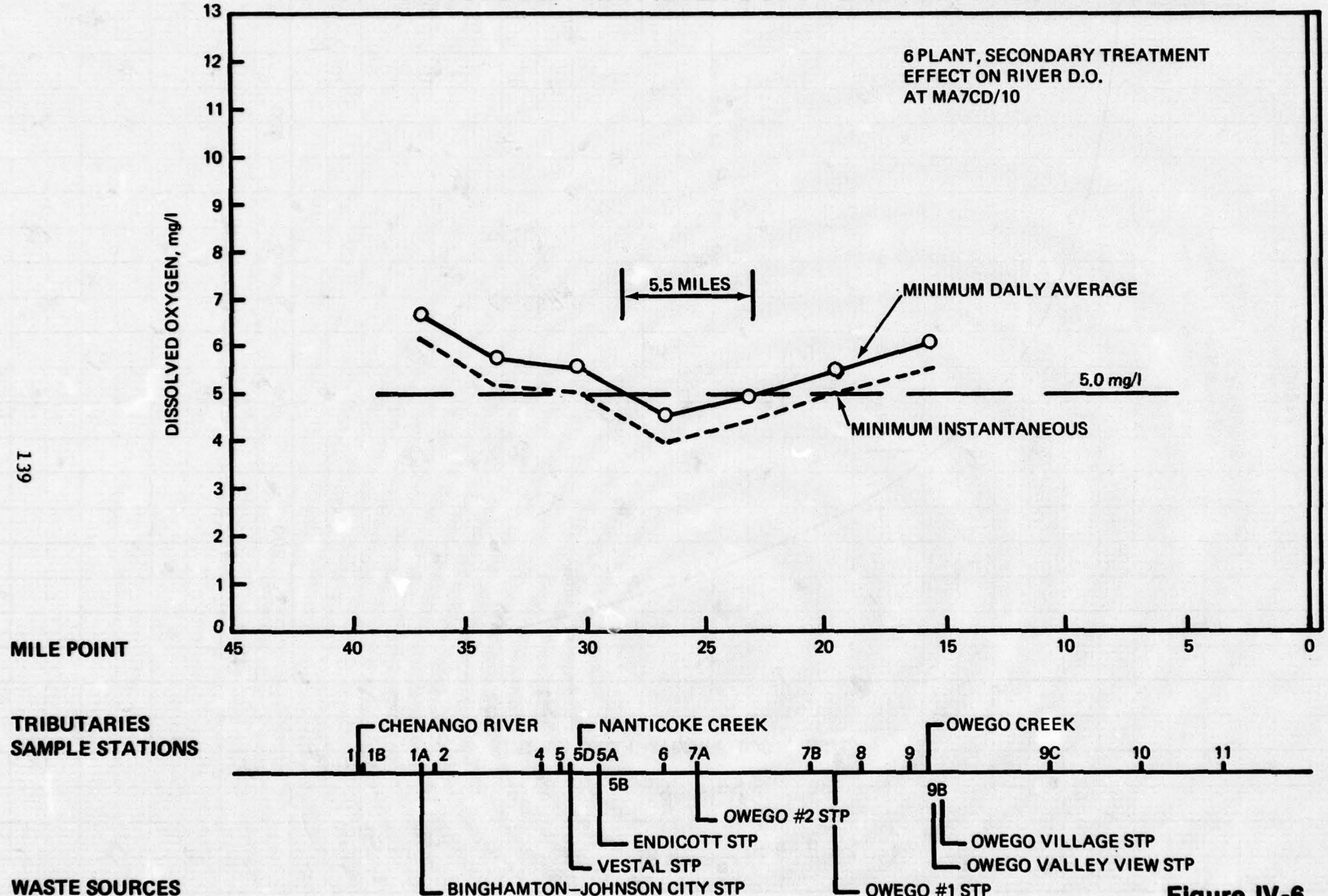


Figure IV-6

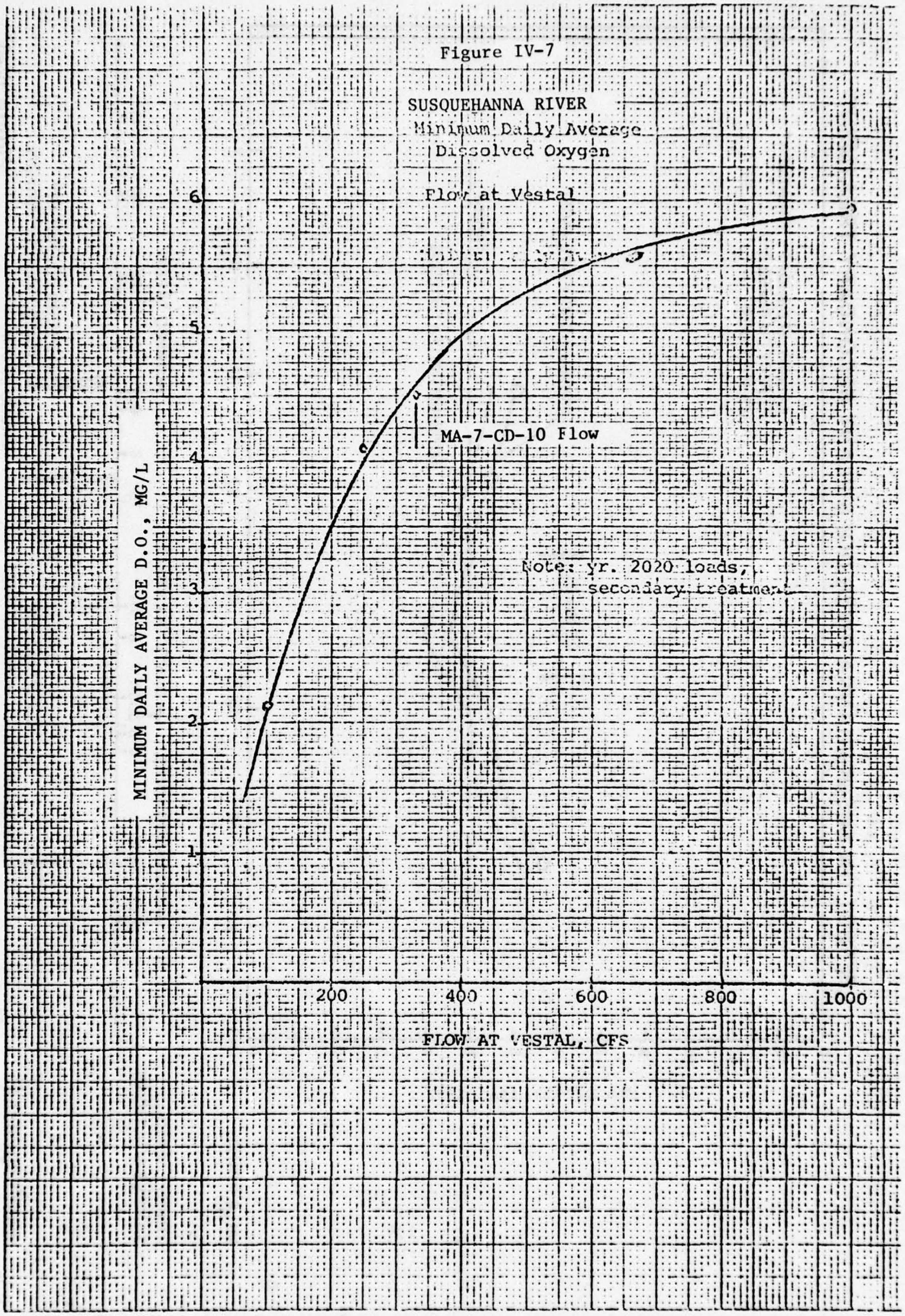


Figure IV-7
SUSQUEHANNA RIVER
Minimum Daily Average
Dissolved Oxygen
Flow at Vestal
MA-7-CD-10 Flow
Note: yr. 2020 loads,
secondary treatment
MINIMUM DAILY AVERAGE D.O., MG/L
FLOW AT VESTAL, CFS
1
2
3
4
5
6
200
400
600
800
1000

Figure IV-8

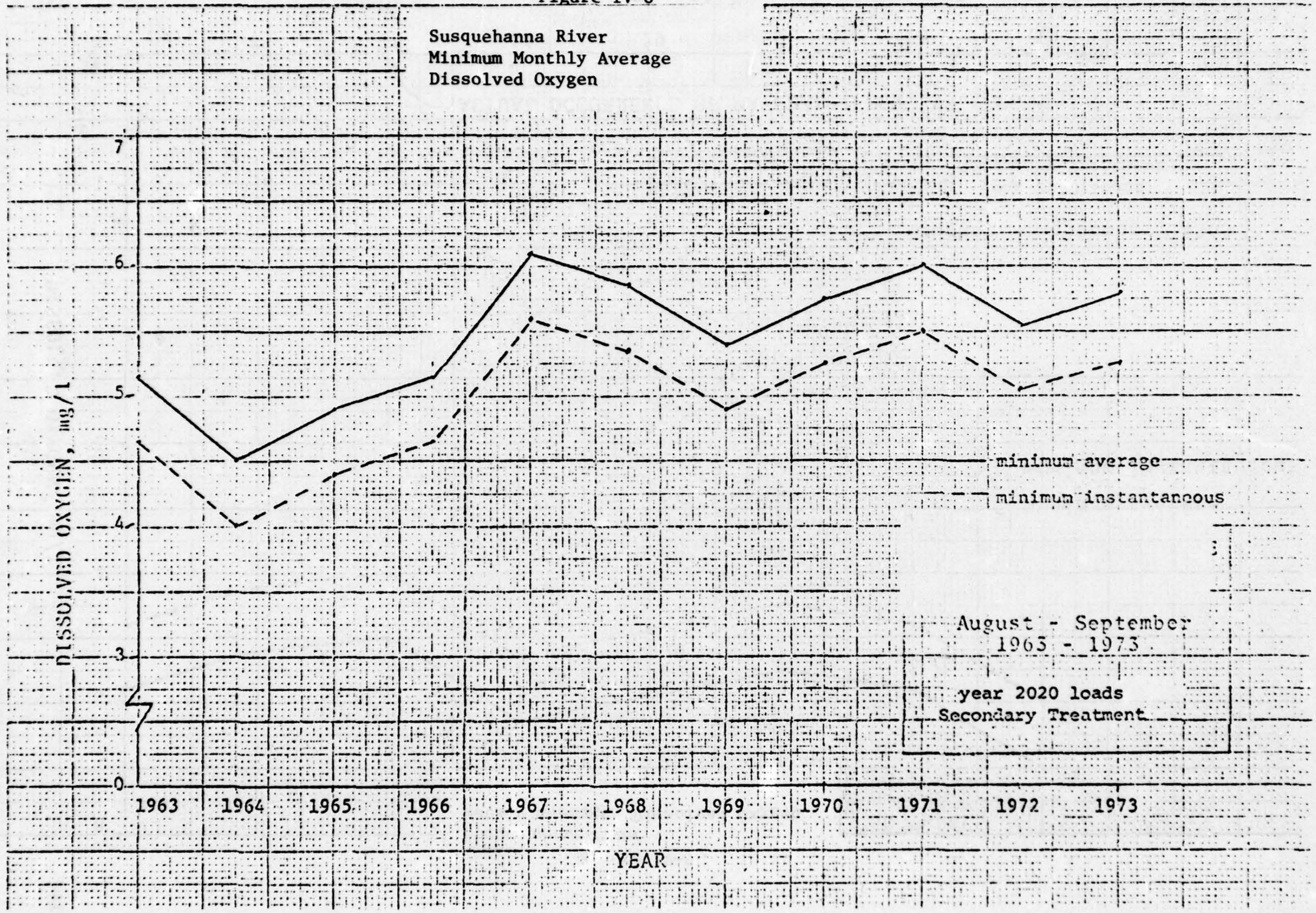
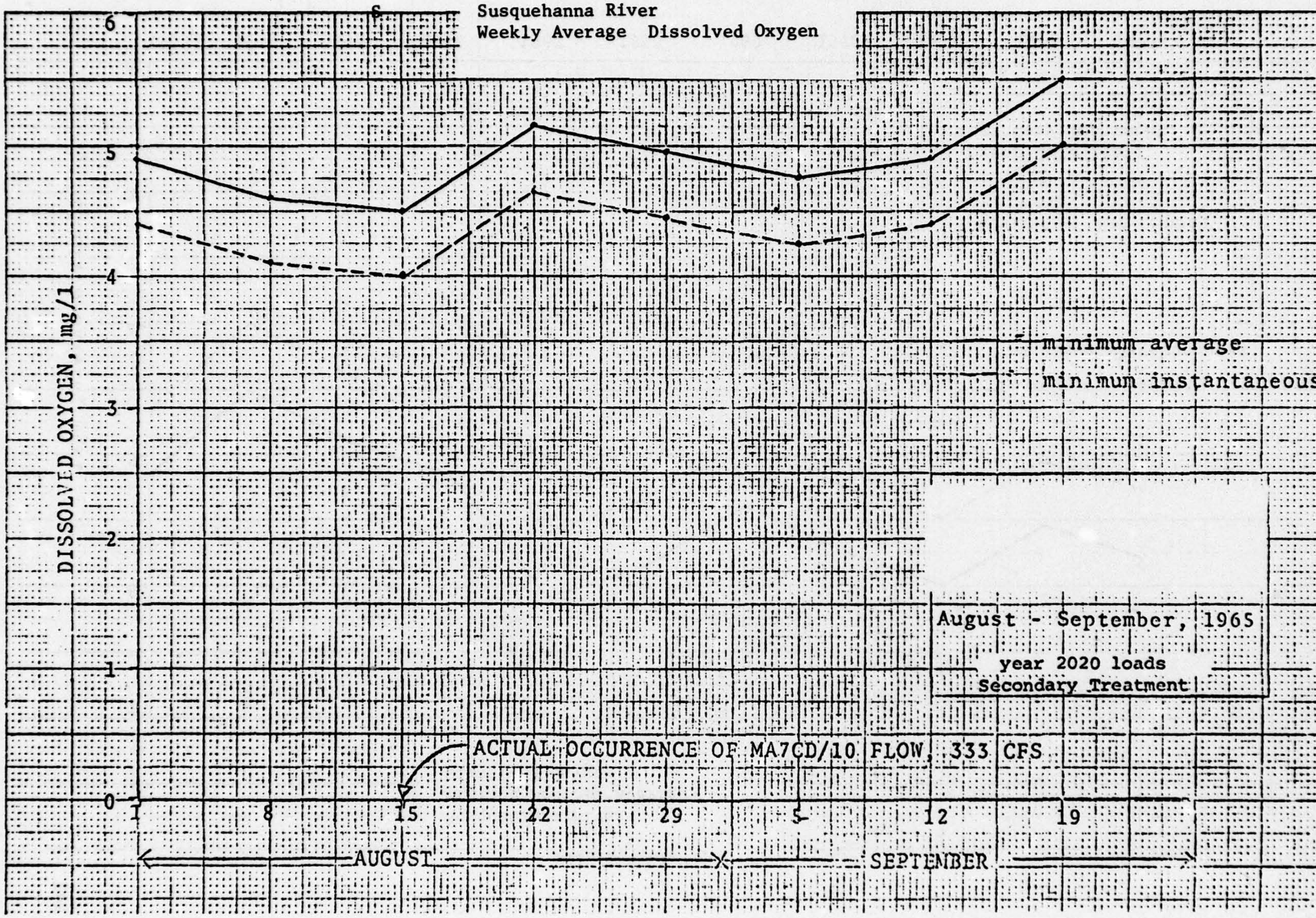

Susquehanna River
Minimum Monthly Average
Dissolved Oxygen
DISSOLVED OXYGEN, mg/L
7
6
5
4
3
0
1963
1964
1965
1966
1967
1968
1969
1970
1971
1972
1973
YEAR
minimum average
minimum instantaneous
August - September
1963 - 1973
year 2020 loads
Secondary Treatment

Figure IV-9
Susquehanna River
Weekly Average Dissolved Oxygen



Both these Figures show, that under these historical flow patterns, the minimum instantaneous DO would be greater than 4.0 mg/l, averaged over the given time period, i.e., monthly for the 11 year period, or weekly for the 1 in 10 year low flow year. The average DO, however, would be less than the 5.0 mg/l standard in 1964 and 1965 for one month average, and in 1965 for three consecutive weeks, with a minimum one week average of 4.5 mg/l.

The use of the model with respect to alternative design has been explained previously in this chapter. An important aspect which arose during the study was the question of whether the 4.0 mg/l alternatives were acceptable, from an aquatic ecology point of view (on which the standards are based). Pertinent to this question is the fact that the model assumes critical conditions in time, and that the low DO does not persist over the entire Susquehanna below Binghamton, but, rather sags to a low value and then recovers. The ISMG requested that the final iteration make an assessment of the biological acceptability of the 4.0 mg/l plans. This was to be done by quantifying in terms of spatial and temporal parameters, the severity of the "low" DO, and using this estimate as a basis for analyzing the probable effect on river fish life.

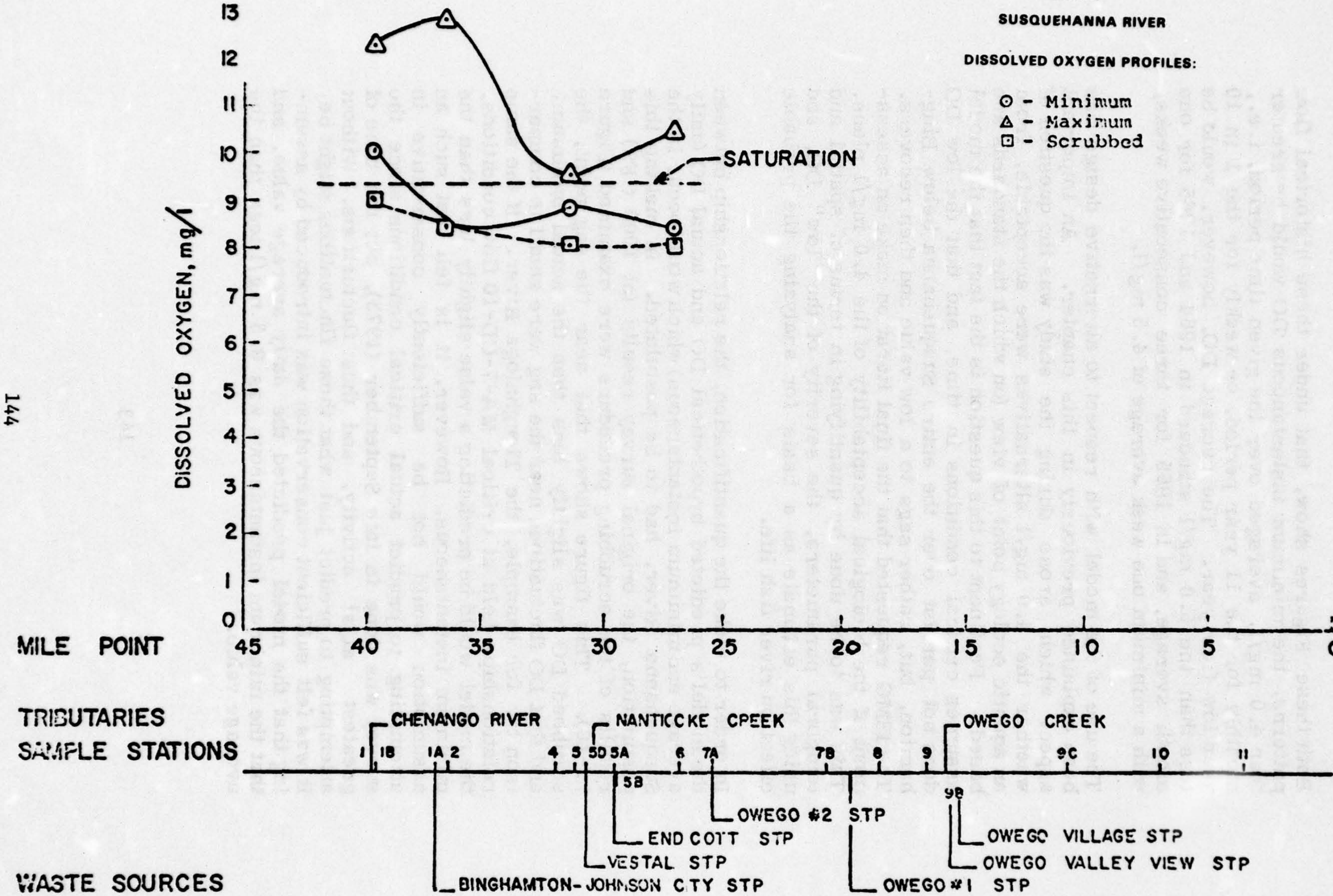
In order to make the quantification, the relationship between the model's predicted hypothetical DO and actual DO (daily average and minimum instantaneous) which would occur in the Susquehanna River, had to be postulated. In making this postulation, the original survey results (at 1000 CFS) and results of the scrubbing procedure were examined (Figure IV-10). This figure shows that near the sag point, the scrubbed DO was slightly less than the actual minimum, and that DO fluctuations near the sag were small in comparison to, for example, the Tioughnioga River. If the same relationships held at critical MA-7-CD-10 flow conditions, the model would be predicting a value slightly less than the minimum instantaneous. However, it is felt that such an assumption would not be sufficiently conservative in attempting to predict actual critical conditions since the survey was done in late September (1973), not the time of greatest algal activity, and thus fluctuations. Without attempting to predict just what those fluctuations might be, it was felt sufficient conservation was introduced by assuming that the model predicted the daily average value, and that the minimum instantaneous was 0.5 mg/l less than the average value.

FIGURE IV-10

Reach 5

SUSQUEHANNA RIVER

DISSOLVED OXYGEN PROFILES:



The question to be answered, given the above information was: Does the 4.0 mg/l plan, i.e., secondary treatment, provide adequate protection for river fish? To put it another way, do short term periods of daily average DO less than 5.0 mg/l over a 5.5 mile reach of river, constitute a threat to fish life? An analysis of this question is discussed in the Impact Assessment Appendix. In summary, the analysis concluded that some investigators have found most fish exhibit some signs of stress when DO drops below 6-7 mg/l, but that the difference in environmental response between 4.5 and 5.0 mg/l, for short periods of time not in the spawning season, is negligible. That is, the secondary treatment plan does afford adequate protection of fish life and meets the intent of the stream standards.

SUMMARY

In summary, the following, combined with the overall Study objectives, were the primary criteria considered in the design of the alternatives:

1. The existing and expected 1977 baseline wastewater management conditions (see Chapter III).

2. Structural and nonstructural flow reduction measures.

3. Federal and State standards.

4. Cost-Effectiveness.

5. Minimizing environmental impact.

In designing wastewater treatment facilities capable of achieving stream standards, and in assessing the impact of the alternatives on the Susquehanna River, a mathematical water quality (DO) model was used. Chapters V, VI, and VII employ the methodology described in this chapter in the analyses of the major technological components: wastewater, sludge, and stormwater management, respectively.

CHAPTER V

DESIGN OF WASTEWATER TREATMENT SYSTEMS

In this chapter, the major detailed technological analyses which defined the wastewater treatment components of the alternatives are documented. These analyses were primarily concerned with the design and cost aspects.

In order not to increase the bulk of this presentation, and to avoid duplication, those analyses which led to applications in many schemes throughout the planning process are not detailed alternative by alternative. Rather, the results of these analytical applications are given in Chapter VIII, the presentation of the final plans. This discussion presents the design basis for both secondary and advanced waste treatment processes. The sludge and stormwater management components are presented in Chapter VI and VII, respectively.

SECONDARY TREATMENT

In the design of secondary treatment facilities, major emphasis was placed on those STP's which had the greatest influence on achieving water quality objectives, i.e., the Binghamton-Johnson City STP and the Endicott STP. The Binghamton-Johnson City plant was important because of its potential use as a regional facility for both the Binghamton-Johnson City and Chenango Valley service areas, the uncertainty with respect to its treatment capabilities, and its size and percentage of the total alternative costs. The Endicott STP was important because of its size, and the fact that there is some uncertainty whether a trickling filter STP can meet secondary treatment standards. This discussion, then, will primarily concern itself with these two STP's, with the other covered in less detail.

BINGHAMTON-JOHNSON CITY STP

This STP was first installed as a primary plant in 1960, and was expanded to secondary treatment in 1973. The nominal capacity of the plant is 18.3 mgd.

In the first summer of the secondary plant's operation, a program was developed by the plant designers and operators to test its capabilities in achieving nitrification, in addition to oxidation of carbonaceous BOD. This program was based on the attainment of unusually high (5000-6000 mg/l) levels of MLSS, and the very high removals of both BOD and NOD, in the order of 90 percent each. The success of this program led to speculation that the plant could be loaded well beyond the 1973 summer levels of about 16-17 mgd, and still achieve secondary treatment. It was felt by many people that the plant actually had a capacity of 25 mgd, rather than the design capacity of 18.3 mgd.

Plant performance was briefly analyzed in Stage II-1 of this Study with the conclusion that, if the high MLSS levels could be maintained at higher flows, its capacity was indeed greater than 18.3 mgd. It was also concluded, however, that its ability to achieve high MLSS at the higher flows was doubtful, and that an analysis of the aeration/clarification system was necessary before a final determination was made.

In Stage II-2, a fairly detailed analysis of the capacity of this STP was made; not only the secondary aeration/clarification system, but all treatment units were examined with respect to present performance and anticipated performance at higher loadings. The result of this analysis was a capacity rating for each major treatment unit. The ratings were used in determining the capital improvements program for the plant for each alternative.

The outcome of this analysis was that the aeration/clarification system has the smallest capacity in this plant. The primary clarification system and sludge handling facilities are adequately sized for well into the future, but the secondary facilities will need expansion immediately (by 1977). Although adequate capacity exists for summer conditions, sludge compaction deteriorates in the winter, resulting in an inability to maintain MLSS levels as high as those observed in the first summer of operation.

The following discussion will first present the analysis of the Binghamton-Johnson City STP and its performance, and then explain the way in which this analysis was incorporated into system design.

Present Wastewater Characteristics

A summary of present wastewater characteristics for the Binghamton-Johnson City area are presented in Table V-1. The data have been divided into the relatively warm period of June through October and the relatively cold period of November through May. June, July, and August 1974 daily data were used to estimate maximum/average ratios. Other pertinent assumptions are noted in the Table. The significant amount of infiltration at this plant (7.5 mgd average) is discussed in Chapter VII.

The treatment plant has relatively high per capita flows and BOD and SS loadings. The influent TSS and BOD_5 were correlated by the equation $BOD_5 = 0.40\ TSS + 54$ with a correlation coefficient of 0.72. The primary BOD and TSS removals and the excess primary TSS and BOD_5 compared to "normal values" have BOD/TSS ratios of 0.32 to 0.40, indicating the high loadings are probably due to normal municipal wastewater materials. The wastewater also is weaker during cold weather, and primary treatment removals are slightly lower.

Influent and primary BOD_5 and TSS were further analyzed for significant relationships. The influent characteristics are given in Table V-2.

TABLE V-2

INFLUENT BOD CHARACTERISTICS

BINGHAMTON-JOHNSON CITY STP

Influent (Raw)	mg/l
Suspended BOD	120
Settleable BOD	71
Non-Settleable BOD	50
Filtered BOD	55
Total BOD	175
% Suspended BOD Settleable	59
% Total BOD Settleable	50

TABLE V-1

PRESENT WASTEWATER CHARACTERISTICS OF BINGHAMTON–JOHNSON CITY STP

Data Base: Jan. 73 to Aug. 74

Estimated Population: 102,600	(June-Oct) Warm	(Nov-May) Cold	All Year
Flow Rates			
Ave. Daily Flow-MGD	17.3	19.2	18.3
Min. Daily Flow Rate-MGD	10.7	8.1	8.1
Max. Daily Flow Rate-MGD	25.6	38.4	38.4
Max. Hourly Flow Rate-MGD[1]	34	38.4	38.4
Ave. Flow Rate, excluding I/I-MGD	10.8	10.8	10.8
Ave. I/I-MGD[2]	6.5	8.4	7.5
Dry weather, low river stage infiltration-MGD[2]	4.2	4.2	4.2
Influent (Raw) BOD			
Ave. BOD-mg/1	206	176	188
Ave. BOD-lbs/day	29,300	29,800	29,600
Ave. BOD-lbs/cap/day	0.286	0.290	0.288
Max. Day BOD-lbs/day[3]	49,800	50,700	50,700
Max. Day BOD-lbs/cap/day[3]	0.49	0.49	0.49
Primary BOD			
Ave. BOD-mg/1	117	110	113
Ave. BOD-lbs/day	16,900	19,300	18,400
Ave. BOD-lbs/cap/day	0.165	0.188	0.179
Ave. Removal Compared to Raw-%	43	37.5	39
Max. Day BOD-lbs/day[4]	35,500	40,500	40,500
Max. Day BOD-lbs/cap/day[4]	0.35	0.40	0.40
Primary NH_3-N			
Ave. NH_3-N-mg/1	17.4	18.2	17.8
Ave. NH_3-N-lbs/day	2,510	3,190	2,890
Ave. NH_3-N-lbs/cap/day	0.0245	0.0311	0.0282
Max. Day NH_3-N-lbs/day[5]	3,760	4,780	4,780
Max. Day NH_3-N-lbs/cap/day[5]	0.037	0.047	0.047
Influent (Raw) TSS			
Ave. TSS-mg/1	353	331	340
Ave. TSS-lbs/day	50,900	58,000	55,300
Ave. TSS-lbs/cap/day	0.496	0.565	0.539
Max. Day TSS-lbs/day[6]	107,000	122,000	122,000
Max. Day TSS-lbs/cap/day[6]	1.04	1.19	1.19

TABLE V-1 Continued

PRESENT WASTEWATER CHARACTERISTICS OF BINGHAMTON–JOHNSON CITY STP

Data Base: Jan. 73 to Aug. 14

	(June-Oct) Warm	(Nov-May) Cold	All Year
Primary TSS			
Ave. TSS-mg/l	113	147	132
Ave. TSS-lbs/day	16,300	25,700	21,500
Ave. TSS-lbs/cap/day	0.159	0.250	0.210
Ave. Removal Compared to Raw-%	68	56	61
Max. Day TSS-lbs/day[7]	35,900	56,500	56,500
Max. Day TSS-lbs/cap/day[7]	0.35	0.55	0.55
Primary Removals			
Ave. BOD removal-lbs/day	12,400	10,500	11,200
Ave. BOD removal-lbs/cap/day	0.121	0.102	0.109
Ave. TSS removal-lbs/day	33,800	32,900	33,300
Ave. TSS removal-lbs/cap/day	0.33	0.32	0.32
Max. Day TSS removal-lbs/day[8]	105,000	102,000	105,000
Max. Day TSS removal-lbs/cap/day[8]	1.02	0.99	1.02
Ratio, Ave. BOD removal: Ave TSS removal	0.36	0.32	0.33
Excess Influent (Raw) Loadings			
Expected Ave. BOD-lb/cap/day	0.17	0.17	0.17
Expected Ave. TSS-lb/cap/day	0.20	0.20	0.20
Excess Ave. BOD-lb/cap/day	0.12	0.12	0.12
Excess Ave. TSS-lb/cap/day	0.30	0.36	0.34
Ratio, Excess BOD Excess TSS	0.40	0.33	0.35
Oxygen Requirements[9]			
Ave. O_2 Requirements-lbs/day	28,200	20,700	–
Max. Day O_2 Reqts-lbs/day	52,400	42,700	52,400.
Sludge Handling Requirements[10]			
Ave. Sludge Handled-lbs/day	42,200	42,600	42,500
Max. Day Sludge Handled-lbs/day	123,000	122,000	122,000

Notes: 1 Max. Hourly flow rate estimated to be 1.33 time Max. day flow rate.
2 Inflow/Infiltration – See Chapter VII
3 Max/Ave. factor of 1.7 estimated from June, July, Aug 74
4 Max/Ave. factor of 2.1 estimated from June, July, Aug 74
5 Max/Ave. factor of 1.5 estimated from June, July, Aug 74
6 Max/Ave. factor of 2.1 estimated from June, July, Aug 74
7 Max/Ave. factor of 2.2 estimated from June, July, Aug 74
8 Max/Ave. factor of 3.1 estimated from June, July, Aug 74
9 Warm: 1.0 × BOD primary + 4.5 NH_3-N primary
Cold: 1.0 × BOD primary + 4.5 NH_3-N primary × 0.1
10 TSS raw – TSS primary + 0.5 × BOD primary

The correlation between primary BOD_5 and TSS was not significant.

Plant Performance

The performance of the Binghamton-Johnson City treatment plant is summarized in Table V-3 for warm and cold months. The averages indicate generally satisfactory performance particularly with respect to BOD but problems have existed for specific time periods. The highest monthly average effluent BOD, 30 mg/l, occurred in November 1973. Monthly average effluent suspended solids were 43 to 191 mg/l during the October 1973 to January 1974 period, and the cell detention time reached a low of 1.5 days in December 1973, while the sludge volume index reached 132 in November 1973.

This plant has accomplished a high degree of nitrification during warm months. The capacity of the plant for future nitrification is discussed in a subsequent section.

The aeration capacity of the plant has far exceeded requirements. Operating experience indicates that operation of only one compressor (out of a total of 3 plus 1 standby) suffices most of the time, while more than two compressors are never needed. Moreover, throttling of the air delivery system is usually constrained by the minimum air flow requirements of the diffusers rather than DO limitations, so that DO is often above the set point (nominally 2 mg/l).

Hydraulic Capacity.

The hydraulic capacity of the present plant is 50 mgd (Clinton Bogert, 1967). Since the estimated ratio of maximum hourly to average flow rate is 2.0, the hydraulic capacity of the plant can be estimated to be about 25 mgd on an average flow basis. However, the peak/average ratio will be affected by interceptor capacities and storage and return to sewers of overflows during storm periods. These considerations raise the hydraulic capacity above 25 mgd, on an average flow basis.

Raw Wastewater Pumping.

The present pumping stations can handle maximum flows of approximately 36.5 mgd (Clinton Bogert, 1967). Hence the capacity of the plant with respect to this area is only 18.3 mgd, on an average flow basis.

TABLE V-3
BINGHAMTON-JOHNSON CITY PERFORMANCE SUMMARY

		Warm (June-Oct)	Cold (Nov-Mar)	All Year
Flows:	Q_{in}	17.3	19.2	18.3
(MGD)	Q_r	11.7	10.4	
	Q_w	0.064	0.063	
Solids	SS_{in}	353	331	340
(mg/l)	SS_{pri}	113	147	132
	Pri. Removal,%	68	56	
	SS_{out}	28	67	46
	MLSS	4,840	3,190	
	MLVSS	3,150	est.2070	
	% Volatile	65	65	
	SVI	77	89	
	$SS_{underflow}$	11,300	8,400	
Growth Rate	Temp, °F	70.4	54.5	
Parameters	pH	7.0	7.1	7.1
	θ_c, days	12.0	4.1	
	μ_c, days^{-1}	0.083	0.245	
	$R_{u, BOD}$, days^{-1} (MLVSS basis)	0.231	0.321	
	T, days = V/Q_{in}	0.140	0.120	
	F/M, d^{-1} (MLSS basis)	0.18	0.25	
	F(N)/M, days^{-1} (MLSS basis)	0.028	0.060	
BOD	BOD_{in}	206	176	188
(mg/l)	BOD_{pri}	117	110	113
	Pri. Removal,%	43	38	
	BOD_{out}	16	18	17
Nitrogen	NH_3 N_{pri}	17.4	18.2	17.8
(mg/l)	NH_3 N_{out}	4.6	14.9	9.7
	NO_3 N_{out}	10.8	1.5	7.3
Sec. Clarifier	$Q_{overflow}$ gpd/ft^2	759	967	856
	$Q_{underflow}$ gpd/ft^2	521	461	498
	Solids Loading, lbs/d/ft^2	48	30	
Misc.	Raw-Pri. TSS, lbs/d	33,800	32,900	33,300
	Raw BOD lbs/d	29,300	29,800	29,600
	Pri. - Final ΔBOD lbs/d	14,500	12,600	13,600
	Pri. NH_3 N as NOD, lbs/d	11,600	15,700	13,700
	Pri. - Final ΔNOD lbs/d	8,340	2,830	

Data Base: Jan. 73 to Aug. 74 for flow, influent and primary BOD and SS
June 73 to Aug. 74 for all others; April & May 74 deleted except for influent and primary parameters.

TABLE V-3 Continued

BINGHAMTON JOHNSON CITY PERFORMANCE SUMMARY

Abbreviations for Table V-3:

Q_{in}	Influent flow rate
Q_r	Sludge recycle rate
Q_w	Sludge wasting rate
SS_{in}	Raw waste suspended solids
SS_{pri}	Primary clarifier effluent suspended solids
SS_{out}	Final effluent suspended solids
MLSS	Mixed liquor total suspended solids
MLVSS	Mixed liquor volatile suspended solids
SVI	Sludge volume index
$SS_{underflow}$	Recycle sludge suspended solids
Temp	Temperature
θ_c	Cell detention time
μ_c	Inverse of cell detention time
R_u	First order reaction coefficients for BOD removal
t	Aeration tank detention time, on STP influent flow basis
F/M	Food: Mass ratio, lbs BOD/lb MLSS or MLVSS

Primary Sedimentation.

The present primary settling tanks have a rated design capacity of 18.25 mgd according to normal criteria. However, excessive overflow rates will at worst reduce process efficiency. It was estimated from the experiences of the plant that only a 10 percent deterioration in process performance would be incurred at average overflow rates of 1200 gal/SF/day (Sewage Treatment Plant Design, WPCF). Actual plant operating data indicated suspended solids removals at 1400 gal/SF/day, nearly equal to those at 1000 gal/SF/day (see Figure V-1). Hence, the capacity of existing primary sedimentation was estimated to be about 25 mgd, on an average flow basis.

Aeration Capacity.

The present aeration capacity is 36,000 cfm with 3 of 4 blowers operating (576 lbs O_2 per minute, or $0,83 \times 10^5$ lbs/day). The maximum daily oxygen transfer requirements have been estimated to be 56,000 lbs/day at year 2020, for 27.4 mgd average flow. This would require up to 2.8×10^6 lbs O_2 per day at 2 percent transfer efficiency and 0.56 and 10^6 at 10 percent transfer efficiency. Plant records have indicated a transfer efficiency of about 10 percent, with 85 percent delivery of air to the aeration basins. Hence, the aeration capacity exceeds 27 mgd, on an average flow basis.

As a general guideline, at 27 mgd, 1.5 times the average BOD removal in the aeration basins might be 42,000 lbs/day. This would require up to 52,500 cfm. At 1200 cfm/lb BOD removal the requirement would be 35,000 cfm. The estimation of a capacity of 27 mgd probably is due to above average transfer efficiency.

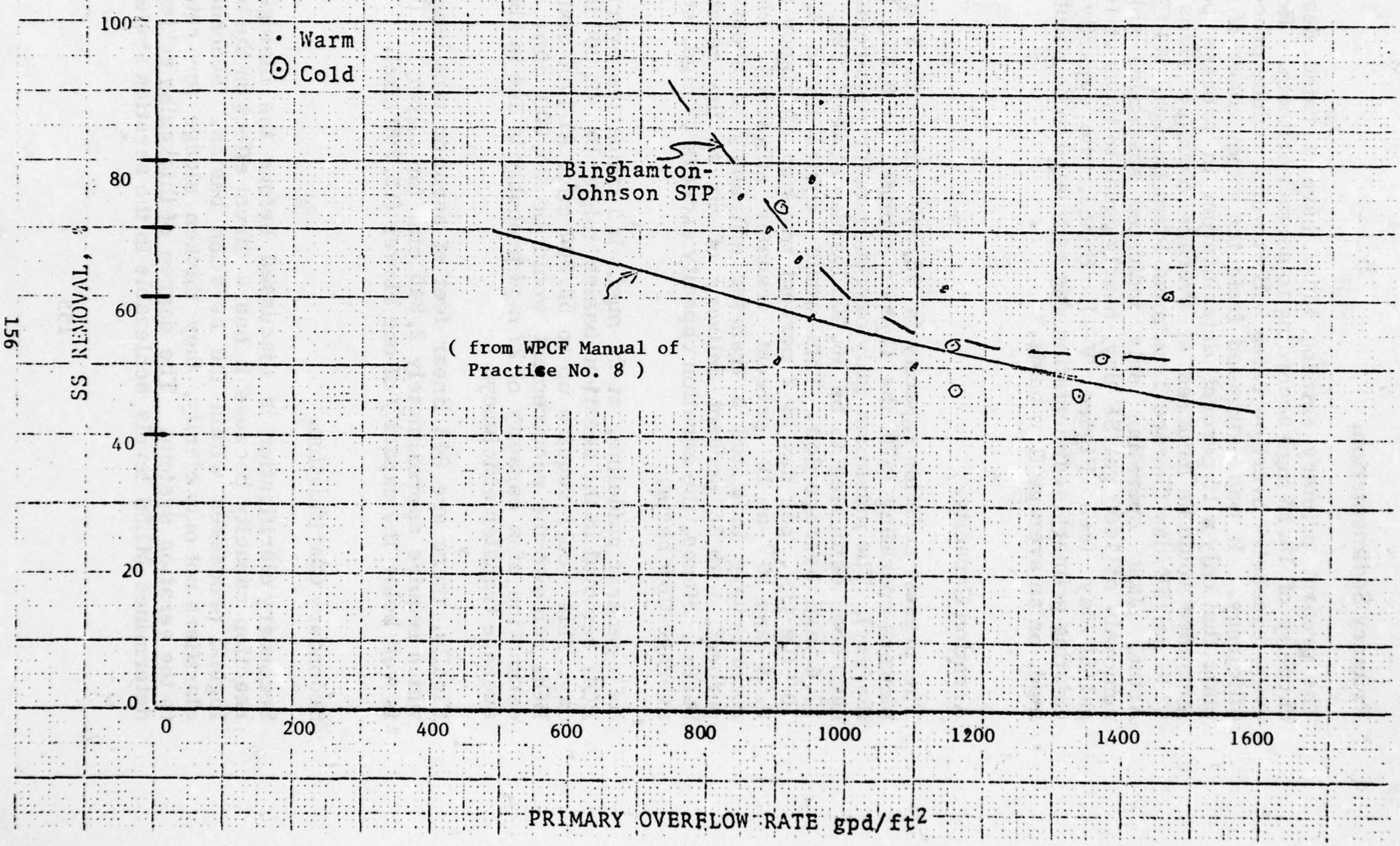
Finally, there are 948 linear feet of aeration tank length, which require approximately 2,840 cfm for mixing. This is well below the capacity of one blower (12,200 cfm).

Secondary Clarification.

Secondary clarification is discussed before the biological reaction capacity because it has a direct effect on the biological processes within the aeration basins. Secondary clarifiers not only clarify, they thicken sludge for return to the aeration system. The degree of thickening achieved determines MLSS levels achievable in the aeration system.

Figure V-1

PRIMARY CLARIFIER PERFORMANCE : Binghamton - Johnson City STP
JUNE 1973 to AUGUST 1974



In general, thickening can be related to underflow velocities. Actual and calculated plant data were plotted in Figure V-2 to illustrate this relationship for both cold and warm periods. Two idealized lines, one for each period, have been drawn through data points. These relationships can be used to predict maximum MLSS achievable at any influent flow rate by a simple mass balance:

$$\frac{Q_{in}\ SS_{in} + Q_r\ SS_r}{Q_{in} + Q_r} = MLSS$$

This mass balance was used in predictions of plant performance, discussed later in this section.

Figure V-3 illustrates effluent suspended solids as a function of overflow rate in the secondary clarifiers based on monthly averaged data. Satisfactory performance has been achieved at overflow rates up to 1200 gal/SF/day, even in cold weather, and when peak flows produced peak overflow rates greater than 1600 gal/SF/day. The capacity of the secondary clarifiers for clarification was estimated to be approximately 27 mgd. This estimate should be tempered by consideration of peak overflow rates in excess of the 1600 gal/SF/day experienced to date. For a 2:1 ratio of peak to average flows, 1600 gal/SF/day corresponds to only 18 mgd. The periods of high effluent SS shown on Figure V-3 were assumed to be the result of biological factors rather than physical ones such as overflow rate.

Biological Reaction Capacity.

BOD and TSS Removal. The performance of activated sludge plants is often assessed in terms of organic loading, or F/M ratio. It is normally assumed BOD and SS removals will be approximately 90 percent each when the F/M ratio is less than 0.6 (based on MLSS), for completely mixed activated sludge systems. On this basis, the capacity of the plant can be estimated to be 25 mgd for the case of inc easing per capita flows and about 19 mgd for the case of constant per capita flows.

Both estimates are based on cold weather constraints, and the lower flow value agrees well with actual experience (up to 30 mg/l effluent BOD at average cold weather flow of 21 mgd).

Figure V - 2
UNDERFLOW-TSS
VS
UNDERFLOW RATE

Binghamton- Johnson City STP

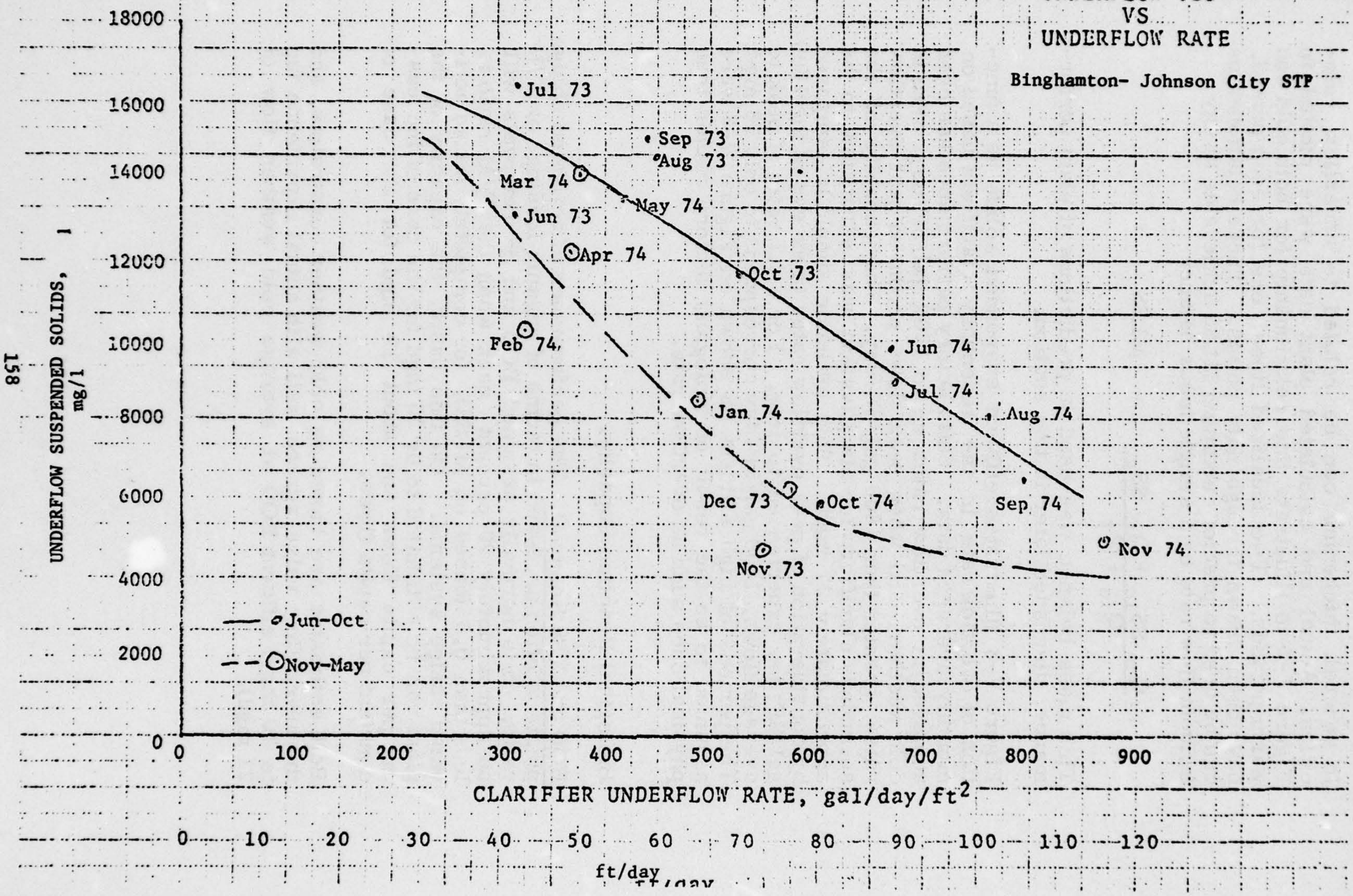
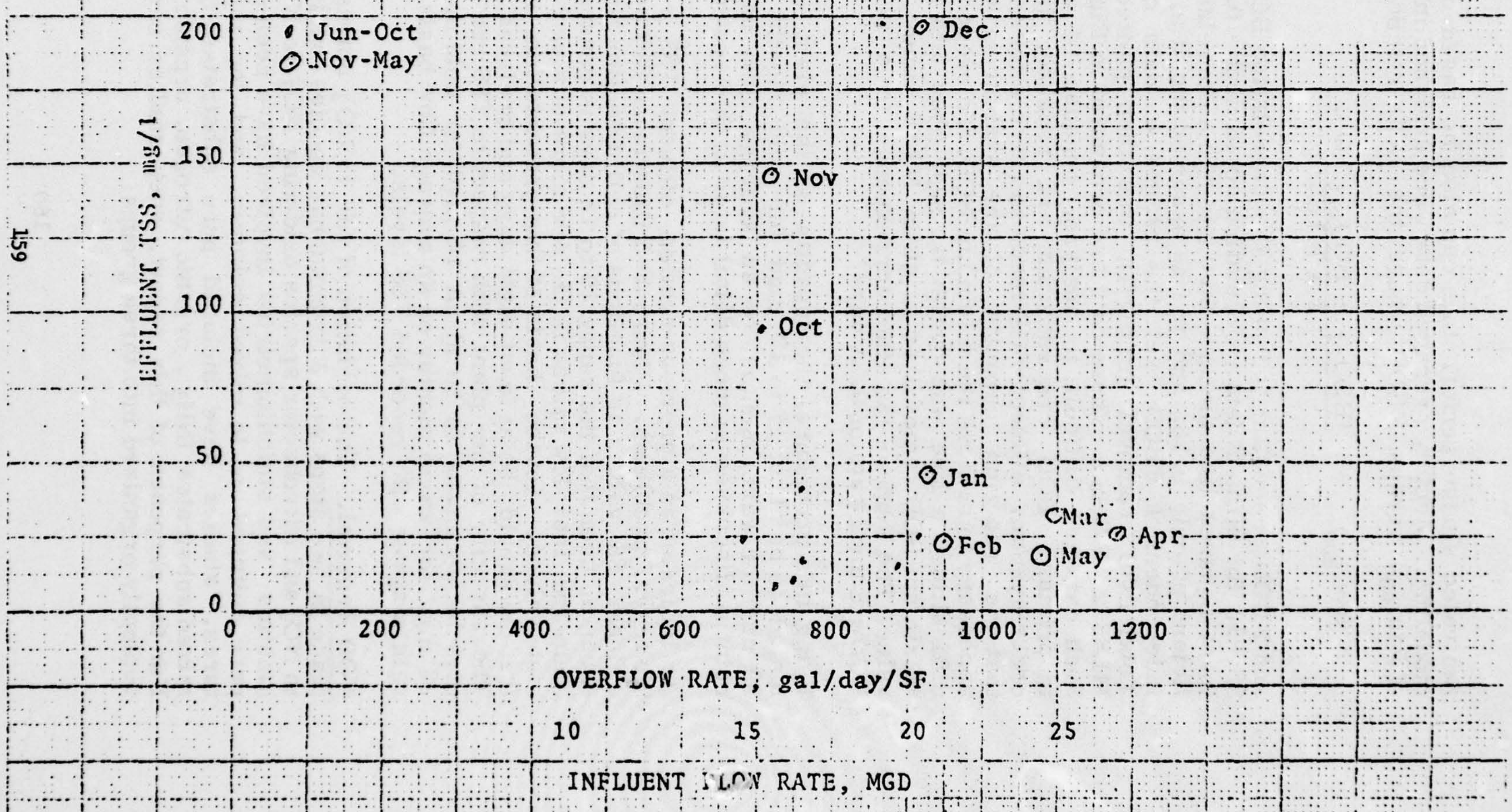


FIGURE V-3
EFFLUENT TSS
vs.
CLARIFIER OVERFLOW RATE

Binghamton-Johnson City STP



Activated sludge performance can also be based on kinetic modeling. Monthly averaged data were analyzed in the form of $R(u,_{BDO})$ versus FBOD where the BOD removed (R):

$$R(u,_{BDO}) = \frac{[BOD_{pri} - BOD_{eff}]\, Q_{in}}{V \times MLSS}$$

and FBOD was estimated using the correlation BOD = 0.20 SS + 10 for effluent data (correlation coefficient = 0.78). A least-squares line through this data had a correlation coefficient of only 0.18, and it was assumed the $R(u,_{BDO})$ was independent of FBOD over the ranges of data ovserved. Total BOD removal (in mg/l), was then plotted against MLSS x V/Q_{in}. These data are presented in Figure V-4. Such data are difficult to analyze, because of variations in primary BOD concentration and because, in a zero order system, effluent BOD tends to reach a minimum such that the reaction capacity of the system exceeds that necessary to achieve the minimum BOD. To overcome these difficulties, a line was drawn through those data indicating maximum BOD removal at a given MLSS times T (days) value. The slope of the line corresponds to a zero order BOD removal rate of 0.31 days $^{-1}$.

Using this procedure, the capacity of the plant was estimated to be about 17 to 19 mgd for cold months and 25 to 29 mgd for warm months, on an average flow basis. This point is discussed in more detail later in this section.

As indicated in a previous section, effluent TSS respond to biological problems. Figure V-5 illustrates SVI values and effluent SS values as functions of cell detention time, O_c. It can be seen that the range 10 to 18 days is generally optimum and that low values of O_c are more of a problem than high values. On the basis of this presentation, it appears the O_c should be at least eight days and preferably higher. The capacity of the plant with respect to TSS removals can thus be estimated to be about 16 mgd for cold months and 22 mgd for warm months on an average flow basis. These values agree with recorded TSS levels.

NOD Removal. The oxidation of NH_3 to NO_3^- normally takes place in two steps with Nitrosomonas species oxidizing NH to NO_2^- and Nitrobacter species oxidizing NO_2^- to NO_3^-. The kinetics and stoichiometrics involved have been studies in pure cultures and in mixed cultures, and for mixed cultures, studies have included pure substrates (NH_3) and mixed substrates (NH_3, organic nitrogen, organic carbon). For the purposes of this report, such studies can be conveniently organized into three groups:

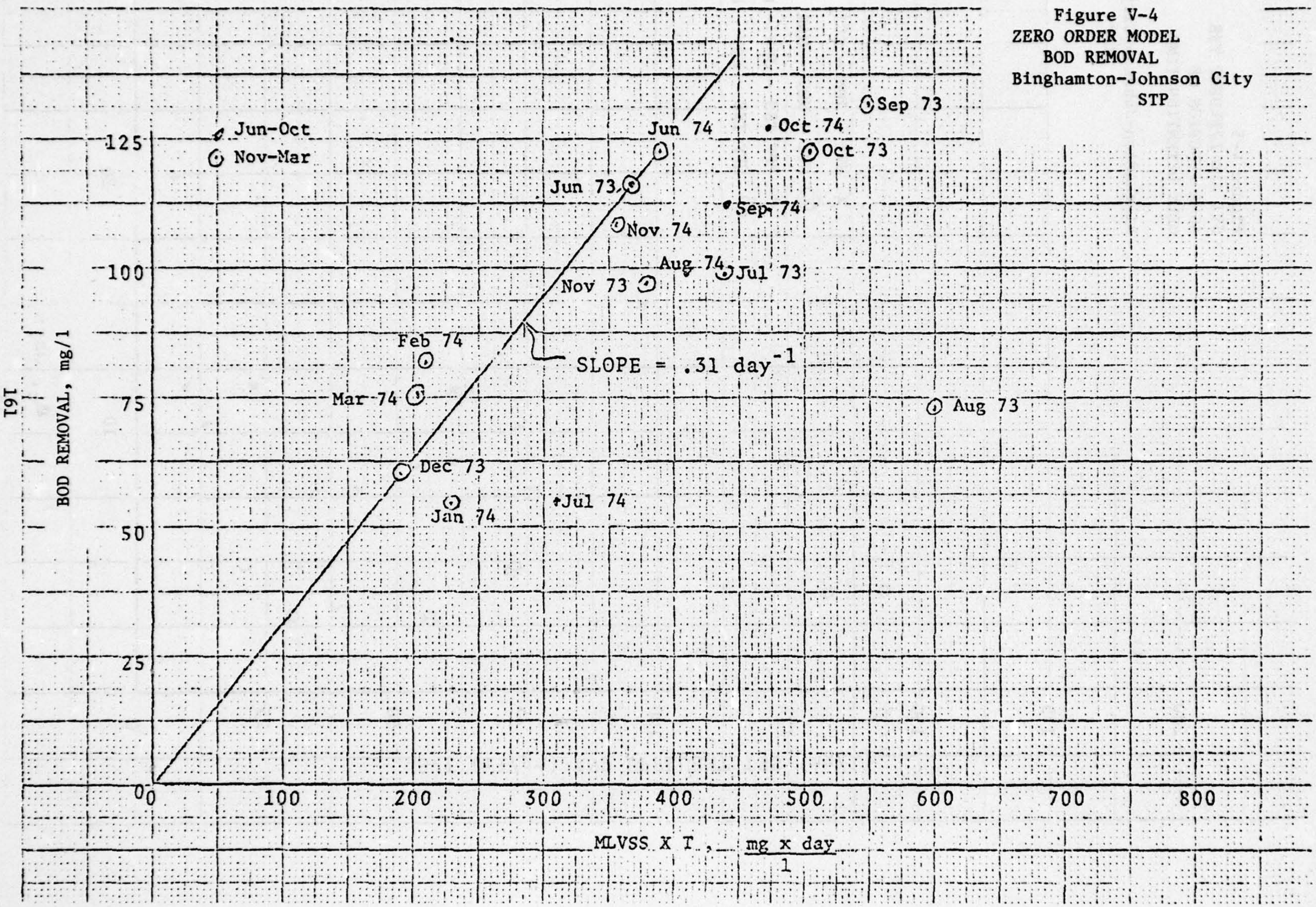


Figure V-4
ZERO ORDER MODEL
BOD REMOVAL
Binghamton-Johnson City
STP
Jun-Oct
Nov-Mar
BOD REMOVAL, mg/1
125
100
75
50
25
0
0
100
200
300
400
500
600
700
800
MLVSS X T , mg x day / 1
SLOPE = .31 day^{-1}
Sep 73
Oct 74
Oct 73
Jun 74
Jun 73
Sep 74
Nov 74
Aug 74
Jul 73
Nov 73
Feb 74
Mar 74
Aug 73
Dec 73
Jan 74
Jul 74

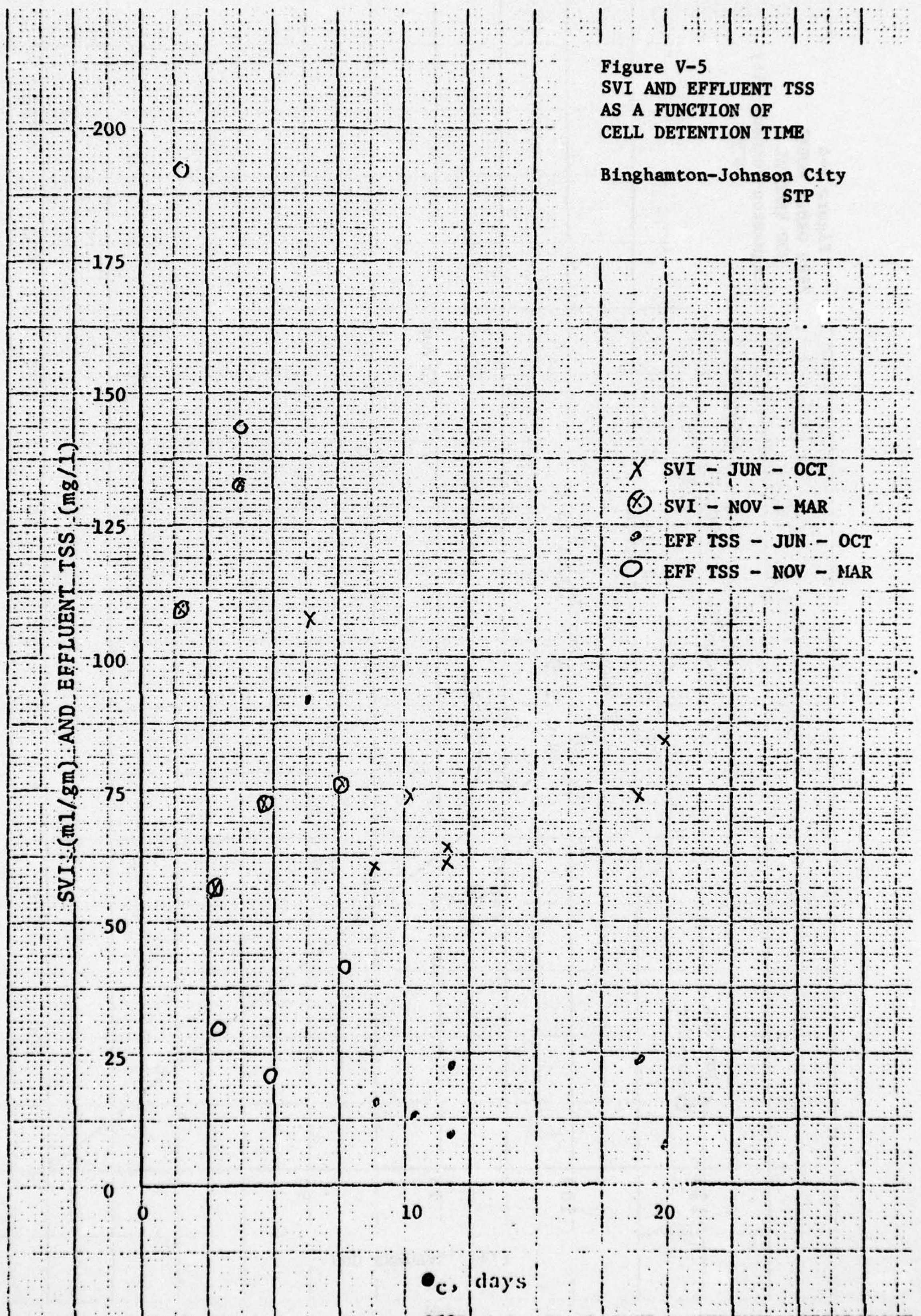


Figure V-5
SVI AND EFFLUENT TSS
AS A FUNCTION OF
CELL DETENTION TIME
Binghamton-Johnson City
STP
SVI - JUN - OCT
SVI - NOV - MAR
EFF TSS - JUN - OCT
EFF TSS - NOV - MAR
SVI (ml/gm) AND EFFLUENT TSS (mg/l)
200
175
150
125
100
75
50
25
0
0
10
20
θc, days

1. Pure or mixed culture, pure substrate studies;

2. "Two sludge" activated sludge systems treating municipal sewage, with nitrification occurring in the second stage;

3. "One sludge" activated sludge systems treating municipal sewage, with oxidation of organics and NH_3 occurring concomitantly.

Typically, studies in Group 1 have found Michaelis-Menton kinetics to hold, while studies in Groups 2 and 3 have found zero-order (with respect to NH_3) kinetics to hold. Studies in Group 1 usually separate the reactions into the two steps mentioned above, while those in Groups 2 and 3 usually identify an essentially direct conversion of NH_3 to NO_3^-. Organic nitrogen is often inadequately reported and will not be considered in the rest of this discussion. Because of the nature of the systems, a gradient in unit NH_3 oxidation rates (dNH_3) is expected, with highest values occurring in Group 1 (dtxMLVSS) and lowest values occurring in Group 3.

Downing, et al (1964) have given the temperature dependence of U_{Max}(nitrification velocity) as:

$$U_{Max}(T) = U_{Max}(15 \text{ degrees C})\ 1.127^{(T-15)}$$

The pH also affects the rate of NH_3 oxidation. Optimum pH values may be in the range 8.0 to 8.6. However, pH was not considered in the above data, except that values were for pH values close to 7.0, as are those for the Binghamton--Johnson City plant.

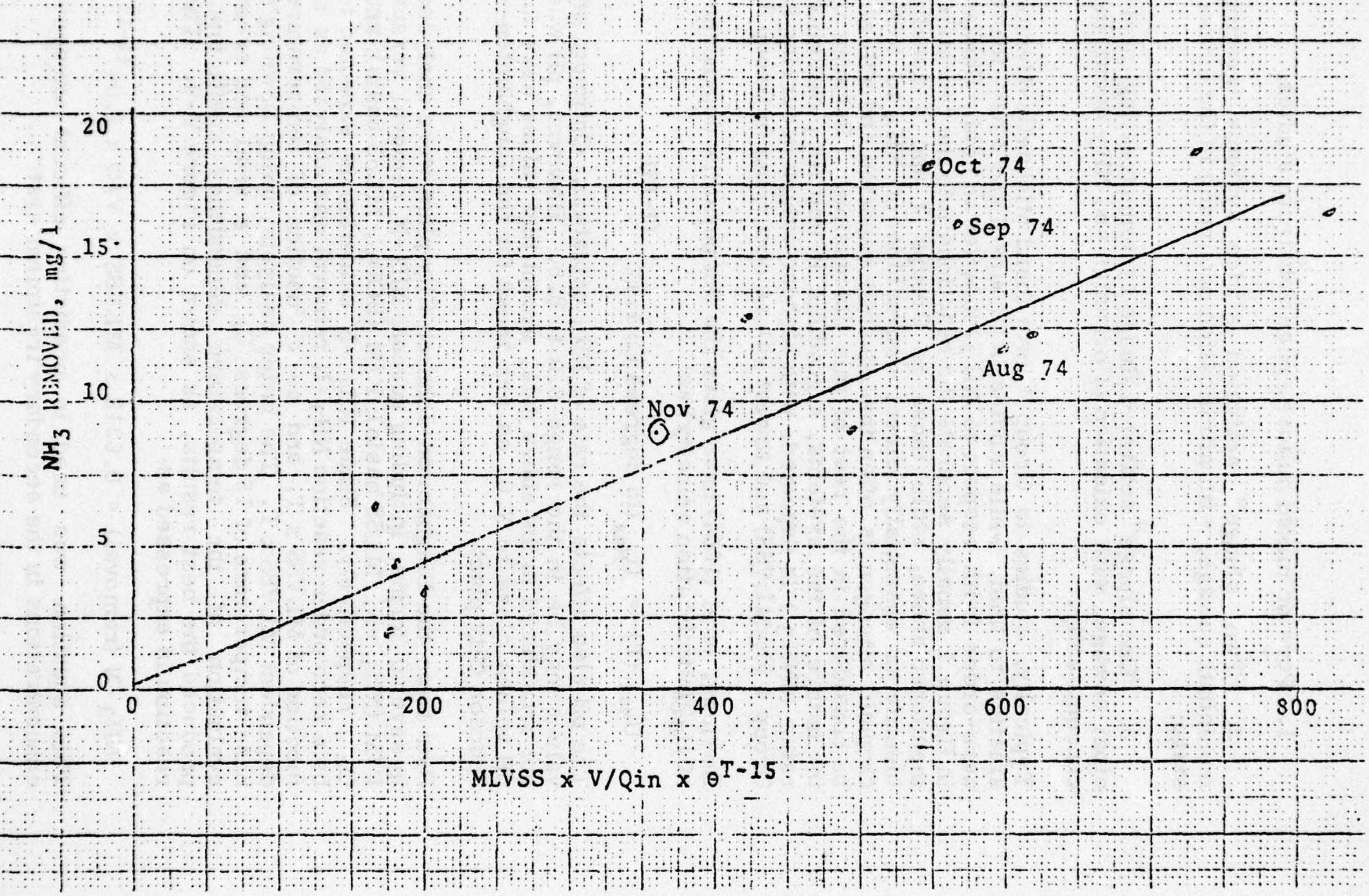
Data from the Binghamton-Johnson City plant were analyzed in several forms, including percent NH_3 -N removal versus F/M (NH_3 -N, MLSS basis); F/M (BOD_5, MLSS basis); and O_c , respectively. Since NH_3 -N oxidation is expected to be a zero order reaction NH_3 -N removal was analyzed as a function of MLVSS x T, and as a function of temperature dependent MLVSS x T. The latter analysis, using Downing's base temperature of 15 degrees C, and a trial and error evaluation of the temperature dependency coefficient, produced the best results, as shown on Figure V-6. The reaction is expressed as:

$$NH_3\text{-N (removal)} = 0.0218 \times \text{MLVSS} \times V/Q \times 1.04^{(T-15)}$$

This equation was used in predicting effluent ammonia concentrations in the secondary treatment system.

FIGURE V-6
AMMONIA REMOVAL AS A FUNCTION
OF MLVSS, TIME AND TEMPERATURE

Binghamton-Johnson City STP



Sludge Processing.

Thickening. Quantities of sludge calculated for the Binghamton-Johnson City plant are given below in Table V-4.

TABLE V-4

RAW SLUDGE GENERATED AT

BINGHAMTON-JOHNSON CITY STP

	Average	Maximum/Day
Primary Sludge, lbs/day	33,300	105,000
Secondary Sludge, lbs/day	6,040	20,000
Total Sludge, lbs/day	39,300	125,000

The value for secondary sludge was based on the sludge wasting flow rate multiplied by the underflow suspended solids.

There are three sludge thickeners, one with an area of 960 square feet and two with an area of 1,960 square feet each. With all three thickeners in operation, the average loading equals the design criterion of 8 lbs/day/SF. During maximum sludge production days, excess sludge can probably be stored in the primary clarifiers, and the thickeners can be operated at slightly higher loadings; also, primary sludge may be fed directly to the digesters. The capacity of the sludge thickeners depends heavily on TSS loadings in the plant influent. Two hypothetical cases are considered in Table V-5, corresponding to the expected range of influent SS. If a 20 percent increase in solids loading is possible, i.e., 9.6 lbs/day/SF, the plant capacity can be estimated to be 24 to 26 mgd. Even at that average loading, the loading with one large thickener out of operation would still be only 16 lbs/day/SF, which is the upper limit of recommended loadings given by "Sewage Treatment Plant Design" (WPCF, 1959).

Therefore, since the maximum flow for any alternative is only 10 percent higher than this estimate, and since little data is available to make a precise estimate, it has been assumed that the thickeners have a capacity of 27.4 mgd, i.e., they will not require expansion during the planning period for any of the alternatives.

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BINGHAMTON WASTEWATER MANAGEMENT STUDY. DESIGN AND COST APPENDI--ETC(U)
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TABLE V-5

PROJECTED THICKENER LOADINGS

Ave. Q in MGD	Primary Sludge lbs/day		Secondary Sludge		Thickener Loadings lbs/ft^2/day	
	Case 1	Case 2	Case 1	Case 2	Case 1	Case 2
10	19,400	17,000	3,400	3,000	4.7	4.1
15	29,100	25,500	5,100	4,500	7.0	6.1
20	34,000	34,000	6,000	6,000	8.1	8.1
25	38,500	42,500	6,800	7,500	9.3	10.2
30	43,700	51,000	7,700	9,000	10.5	12.3
35	48,200	59,000	8,500	10,500	11.6	14.3

Note: Case 1 corresponds to increasing per capita flows.
Case 2 corresponds to constant per capita flows.

Digestion. The volatility of sludge solids is not known, but if they are 70 percent volatile, which is a normal value for primary sludge (Metcalf and Eddy, 1972), the volumetric loading on the digesters would be only 109 lb VSS/day/CF, a lower range of accepted levels. At 4 percent TSS, detention time is presently 22 days, the upper range of accepted values. It would appear at least a 50 percent increase in loading to the digesters is feasible. In terms of sewage flow rate, it was estimated from Table V-5 that this increased loading represents a plant capacity of about 29 to 36 mgd, on an average flow basis, for Cases 2 and 1, respectively. Therefore, the alternatives did not include expansion of digestion capacity.

Assuming 60 percent digestion of VSS (Metcalf and Eddy, 1972), and conservation of non-volatile solids, the average solids loading to the vacuum filters is about 23,000 lbs/day (seven day per week basis). At the present operating schedule of 70 hours per week, assuming 95 percent solids capture, filter loading is 5.1 lbs/SF/hr, slightly higher than the design loading of 5 lbs/SF/hr (Clinton Bogert, 1967). It was also known the plant experiences difficulty in dewatering properly at these rates, and that a loading of 3.5 lbs/SF/hr is often recommended ("Sewage Treatment Plant Design," WPCF).

To estimate maximum plant capacity for vacuum filtration, it was recognized that, by operating 22 hours per day and seven days per week, operation time could be more than doubled over that presently practiced. Therefore, maximum capacity at present loading rates would be about 37 mgd, and 26 mgd at a rate of 3.5 lb/SF/hr. Normally, however, plants of this size do not operate vacuum filters for this length of time. The choice of whether to expand the filtration capacity by increasing filter size or by extending operation time should be based on internal economics; in any event, the comparative economics of this Study's alternatives would not be affected by an increase in vacuum filtration costs, since O&M costs were increased at Binghamton-Johnson City as service area increased (see below). For the sake of the alternative design, then, it was assumed that the vacuum filtration capacities are 26-37 mgd, and would not require capital expansion to meet the demands of any of the alternatives.

Summary.

The considerations of plant capacity presented in the above sections are summarized in Table V-6. Where appropriate,

TABLE V-6

SUMMARY OF BINGHAMTON-JOHNSON CITY TREATMENT PLANT CAPACITY mgd

Treatment Area	Case 1	Capacity, Average Q_m Basis	Case 2
Hydraulic Capacity		>25	
Raw Wastewater Pumping		18	
Primary Sedimentation		25	
Aeration Capacity		27	
Secondary Clarification		18*-27	
BOD Removal	19-25		17-19
TSS Removal		16	
Sludge Thickening	26		24
Digestion	36		29
Vacuum Filtration	37		26

*For 2:1 Peak to Average Flow Ratio

Note: Case 1 corresponds to increasing service area and increasing per capita flows.
Case 2 corresponds to increasing service area but constant per capita flows.

capacity for each item is given for Cases 1 and 2, where Case 1 corresponds to a combination of future increases in service area and in per capita flow rates, and where Case 2 corresponds to increases in service area but constant per capita flow rates. Both cases assume constant per capita mass loadings.

As indicated in the table, the plant is already at or just beyond capacity with respect to several treatment areas. As flow rate to the plant increases, the plant capacity can be increased by several combinations of increases in separate areas. For future average flow rates 25-27 mgd, for example, a 50 percent increase in secondary clarification capacity would achieve satisfactory BOD, TSS, and warm weather NOD removal. Raw wastewater pumping would also have to be added. For further increases in flow rate, increases in all other areas except possibly digestion would be necessary, including hydraulic modifications throughout the plant.

Design of Alternatives--B-JC STP Secondary Treatment

The previous section described the analysis of the existing B-JC STP and concluded the following units of the plant will require expansion to meet the needs of any of the action alternatives: raw wastewater pumping, aeration and secondary clarification, and primary clarification.

The raw wastewater pumping capacity can be modified quite easily by changing impellers on the existing pumps. This was included in all alternatives which involved increases in flow over those presently occurring at the plant.

Primary clarification expansion would be necessary when the flow reaches 25 mgd. The cost estimates for this expansion assumed this would be done by adding a primary clarifier unit equal in size to the existing units, although this could be impractical, due to the existing piping systems. In the event of such interference, it would also be possible to expand the sedimentation capacity by use of tube settlers, which would have a cost in the same order of magnitude as an additional sedimentation tank.

The secondary treatment capacity expansions were designed on the basis of the relationships developed in the previous section for sludge compaction, MLSS optimization and bio-oxidation kinetics. The following discussion will outline the basic principles applied in these designs and will also demonstrate, by typical example, the computational method

used. Following that, a summary of the expansion designs will be given.

The achievement of secondary treatment at the B-JC STP was basic to all alternatives except Baseline. The alternatives were designed to meet secondary treatment performance, at the secondary clarifier effluent, throughout the planning period, i.e., this effluent was never allowed to deteriorate, even though it was quite possible the nitrification units would remove any carbonaceous material not removed in the secondary system.

Winter conditions controlled the design of the secondary treatment system, but warm weather conditions were critical in the receiving water quality analysis. Thus, the analysis of plant performance was done for both periods. The first step was the design, in which, for a given combination of influent flow and BOD, expansions were computed to meet the standards of 85 percent BOD removal and a maximum effluent BOD of 30 mg/l. For the indicated number of additional aeration units and clarifiers, summer performance for BOD and NOD was computed at benchmark years in the planning period and used as input to the stream model to determine resulting DO. If the target DO was not met by secondary treatment alone, nitrification was added in the year at which the violation was predicted.

The example chosen to demonstrate typical computations is the Baseline Profile, in which the following major assumptions were used:

1. No physical expansion of STP.

2. No flow reduction (structural or nonstructural) achieved.

3. No service to Chenango Valley.

The computation is given in tabular format in Table V-7.

As seen in this computation, the BOD removal in the winter would fall below 85 percent between 1973 and 1980, and the winter effluent BOD would exceed 30 mg/l in the same period.

Thus, for the given (Baseline) flow and BOD projections, expansion of the aeration and clarification system would be necessary to maintain secondary treatment.

TABLE V-7

BINGHAMTON-JOHNSON CITY STP PERFORMANCE BASELINE PROFILE

Identification	YEAR 1973	1980	2000	2020
Population (1000's)	100	116.1	118.7	123.8
Flow (MGD)	18.3	21.6	24.5	25.3
Raw Waste BOD, lbs/cap/day	0.288	0.288	0.288	0.288
Raw Waste BOD, lbs/day	28,800	33,400	34,200	35,700
Raw Waste BOD, mg/l	188	185	167	167
Primary Effluent BOD, lbs/cap/day	0.179	0.179	0.179	0.179
Primary Effluent BOD, lbs/day	17,900	20,800	21,200	22,200
Primary Effluent BOD, mg/l	117	115	104	104
Primary Effluent NH_3-N, lb/cap/day	0.028	0.028	0.028	0.028
Primary Effluent NH_3-N, lb/day	2,800	3,250	3,330	3,480
Primary Effluent NH_3-N, mg/l	18.4	18.1	16.4	16.4
Maximum MLVSS, warm months	3,100	2,750	2,550	2,500
mg/l cold months	2,300	1,950	1,750	1,700
Aeration Time, Days	0.131	0.112	0.099	0.096
MLVSS - Time, warm months	405	308	252	240
mg/l-days cold months	300	219	173	163
BOD removed, mg/l warm months	127	96	78	74
cold months	93	68	54	51
Effluent BOD warm months	15-20*	15-20*	26	30
mg/l cold months	24	47	50	53

** non-removable fraction*

TABLE V-7

BINGHAMTON-JOHNSON CITY STP PERFORMANCE BASELINE PROFILE

Identification		1973	1980	2000	2020
		YEAR			
% BOD removed	warm months	~90	~90	84.5	82
	cold months	87	75	70	68
TEMPERATURE, °F	warm months	70	70	70	70
	cold months	54	54	54	54
TEMPERATURE, °C	warm months	21	21	21	21
	cold months	12	12	12	12
Temp. Correction - 1.04^{T-15}	warm months	1.27	1.27	1.27	1.27
	cold months	0.89	0.89	0.89	0.89
MLVSS - 1.04^{T-15}	warm months	515	390	320	305
	cold months	270	195	155	145
NH_3-N Removed mg/l	warm months	11	8.5	7.0	6.7
	cold months	6	4.2	3.4	3.2
Effluent NH_3-N mg/l	warm months	7.4	9.6	9.4	9.7
	cold months	12.4	13.9	13.0	13.2
Effluent BOD lbs/day	warm months	3,050	3,600	5,300	6,300
	cold months	3,660	8,100	10,200	11,200
Effluent NOD lbs/day	warm months	5,200	8,000	8,800	9,400
	cold months	8,700	11,500	12,200	12,800

* non-removable fraction

For any alternative, computations such as those just given were performed to determine the year or years in which expansion of facilities would be necessary to maintain secondary treatment in the winter. Note that the performance of a system is a function of both influent flow and BOD:

1. As flow rises, aeration time is decreased, and maximum MLVSS is also decreased, resulting in a reduction in the MLVSS--Time product, and, therefore, BOD removal capacity.

2. As BOD increases, a greater removal is required to meet both the 85 percent removal requirement and the 30 mg/l limitation.

After a sufficient number of designs were done in Stage II-2, it was possible to generalize the relationship between the expansion requirements and the influent flow and BOD, as shown on Figure V-7. Using this Figure, and the computed influent flow and BOD (based on population, per capita flow, infiltration control level, level of achievement of nonstructural flow reduction and per capita BOD), over the planning period, the required expansion could be determined. This Figure was used in Stage III to determine the capital expansion programs for secondary treatment at B-JC.

The need for nitrification was derived from the secondary treatment analyses, using the summer performance estimates and the receiving water quality model. The year in which nitrification was estimated to be necessary was that year in which summer effluent from the secondary system, as expanded to meet secondary effluent standards in the winter, resulted in a BOD and NOD which, combined with the Endicott secondary effluent, resulted in a receiving water DO of less than the target DO of 5.0 mg/l.

In Stage II-2, several conditions of loading to the B-JC STP were tested for alternatives whose objective was to achieve 5 mg/l DO in the Susquehanna River:

1. No flow reduction, no service to Chenango Valley.

2. Infiltration control of 4.5 mgd, no service to Chenango Valley.

3. 100 percent nonstructural control, no service to Chenango Valley.

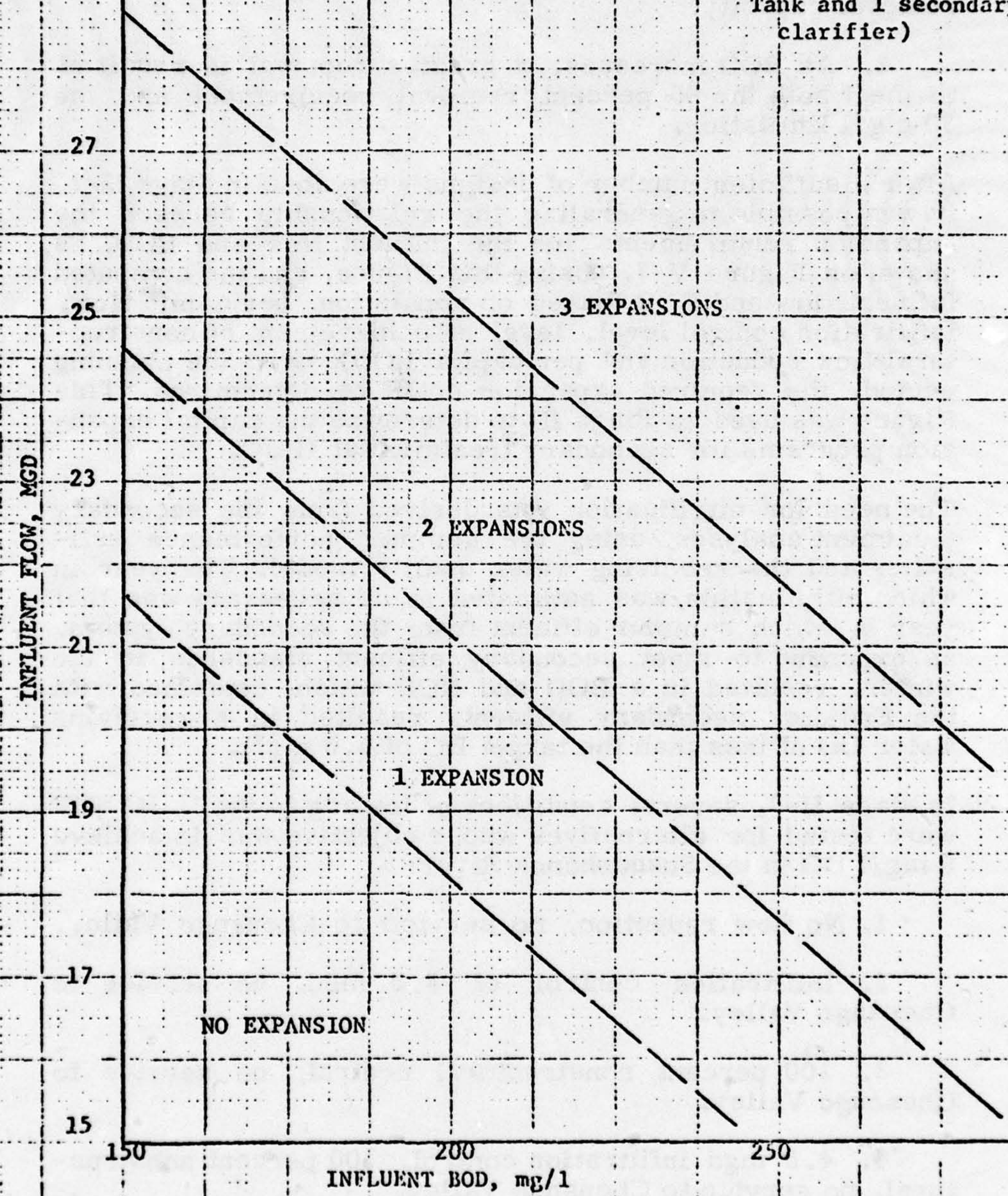
4. 4.5 mgd infiltration control, 100 percent nonstructural, no service to Chenango Valley.

FIGURE V-7

B-JC STP

SECONDARY TREATMENT EXPANSIONS REQUIRED AS A FUNCTION OF INFLUENT BOD AND FLOW

(EXPANSION = 1 aeration Tank and 1 secondary clarifier)



In addition, the effect of serving Chenango Valley at the B-JC STP was investigated with respect to need for nitrification. The initial result of this analysis, using the methodology described above, was to require nitrification at the B-JC STP in the following years:

	Year
B-JC Service Area only, no flow reduction	1983
B-JC & Chenango Valley, no flow reduction	1980
B-JC Service Area only, infiltration control only	2000
B-JC Service Area only, nonstructural control only	2000
B-JC Service Area only, infiltration & nonstructural	2027

It was clear, therefore, the flow reduction measures had a greater "delaying" effect than the shortening effect of adding the Chenango Valley wastes. In addition, to be strictly consistent in comparing the cases of regionalizing Chenango Valley or not, the alternatives with Chenango Valley served at the B-JC STP should have nitrified only a small fraction of the Chenango Valley waste (equivalent to the reduction in oxygen demand of that effluent achieved in the river system with a separate Chenango Valley STP), and by-passed the nitrification units at B-JC STP with the remainder. Economically, the regionalization alternatives would be "penalized" for nitrifying Chenango Valley waste in comparison to the separate treatment scheme which would provide only secondary treatment of this waste. Additionally, the regionalization scheme would have to apply the additional treatment to the B-JC service area waste sooner.

Recognizing that flow reduction measures would have a greater effect on delaying nitrification than would serving the Chenango Valley area separately, it was decided (in the regionalization alternatives), to nitrify the Chenango Valley portion of the waste along with the B-JC waste, in the same year in which nitrification then would take place in the separate treatment schemes. Thus, added costs were involved in that the Chenango Valley area waste received a higher degree of treatment in the regionalization alternatives than in the separate treatment schemes. The need for nitrification was then dependent only on the level of flow reduction achieved.

In Stage III-1, the analyses of the effects of specific levels of flow reduction were expanded to include several levels of reduction, in order to determine the cost effective level of both infiltration control and nonstructural flow reductions. In this analysis, Figure V-8 was a tool used in determining the year in which nitrification would be necessary to main-

FIGURE V-8

Binghamton-Johnson City STP

YEAR NITRIFICATION
REQUIRED FOR 5.0 mg/1 D.O.

LEVEL OF NON-STRUCTURAL
ACHIEVEMENT

INFILTRATION REDUCTION (MGD)	100%	50%	0%
0	2000*	1992	1983*
1	2006	1996	1987
2	2012	2001	1990
3	2018	2006	1994
4	2024	2011	1998
4.5	2027*	2013	2000*
5	2030	2016	2002

* GIVEN

tain 5 mg/l, based on the determination that the year in which nitrification would be necessary was a function of the level of infiltration control and degree to which the nonstructural measures would be achieved. The points for 0 and 4.5 mgd I/I reduction, combined each with 0 percent and 100 percent achievement of the nonstructural measures provided benchmarks (noted by an asterisk) as given above. The time scale was then proportioned to the infiltration reduction scales on the left and right vertical columns; and then the time scale on the center column (50 percent nonstructural achievement) horizontally from the left and right columns.

For example, on the top row (0 infiltration reduction 1992 (50 percent) is midway between 2000 (100 percent) and 1983 (0 percent). In order to insure the accuracy of the assumptions regarding this linear relationship between year for nitrification and flow reduction, the center column was checked to show proportioning from the two end points (1992 and 2016 derived from the horizontal calibration) would result in the same years as the portioning of the horizontal rows from the left and right columns. Thus, on the 50 percent column, the year for nitrification with 2 mgd infiltration reduction, 2001 is midway between 1992 and 2011, the years for 0 mgd and 4 mgd reduction (on the vertical column 50 percent), just as 2001 is midway between 2012 and 1990 on the 2 mgd horizontal row.

The details of the nitrification design itself are discussed later in this Chapter.

O&M Costs, Secondary Treatment at B-JC STP

As detailed in Chapter II, the present (1974) O&M costs at B-JC STP are $685,000 per year when the flow was 18.3 mgd. Chapter II also pointed out the O&M costs for secondary treatment are strongly dependent on design capacity, and not actual flow rates. With reference to the future O&M costs at B-JC STP, then, the fact that most of the treatment units at this plant have a capacity considerably greater than the 1974 flow rates must be taken into consideration.

In addition, future flows could increase gradually, by population growth and inceases in per capita waste flows; or could decrease due to infiltration control; or increase quickly due to connection of sizeable new service areas, e.g., Chenango Valley.

Since the plant generally has a capacity far greater than the 1974 flows, it was projected that O&M costs, in 1974 dollars, would not increase significantly if flows were increased gradually, and, by the same token, would not decrease significantly if flows were reduced. Therefore, for those alternatives which do not expand service to Chenango Valley, the 1974 O&M costs were kept constant for the planning period. With Chenango Valley connected to the plant, the O&M cost was computed (from Figure II-9) as the cost at 18.3 mgd +2.2 mgd (from Chenango Valley), or 20.5 mgd. The cost would be $742,000 per year (1974 dollars). O&M costs for separate treatment at Chenango Valley were computed in increments, corresponding to the initially constructed and future construction design capacities.

ENDICOTT STP

Operating data for this plant, from January 1974 through July 1974, were analyzed with the intention of modeling the performance of the plant for BOD removal, in order to make predictions of plant performance under future conditions of flow and BOD loading resulting from population growth and expansions of service area.

Influent Characteristics

Table V-8 shows the characteristics of the raw waste, by month, as an average of the characteristics of the sampling days, usually once per week.

TABLE V-8

INFLUENT CHARACTERISTICS FOR ENDICOTT STP (1974)

Month	Q MGD	Temp Deg. C	BOD mg/l	BOD lb/day	TSS mg/l	TSS lb/day
January	4.66	9.4	116	4500	116	4500
February	3.94	8.0	116	4050	215	7100
March	4.41	8.6	127	4650	128	4700
April	5.98	8.6	76	3800	129	6450
May	3.75	12.2	139	4350	228	7150
June	3.41	14.7	157	4450	216	6140
July	3.35	16.4	98	2740	139	3880
Average	4.2		117	4080	163	5700

Average/Capital: 94 GPD/Capita

(Population = 44,700) 0.091 lbs/day BOD/Capita

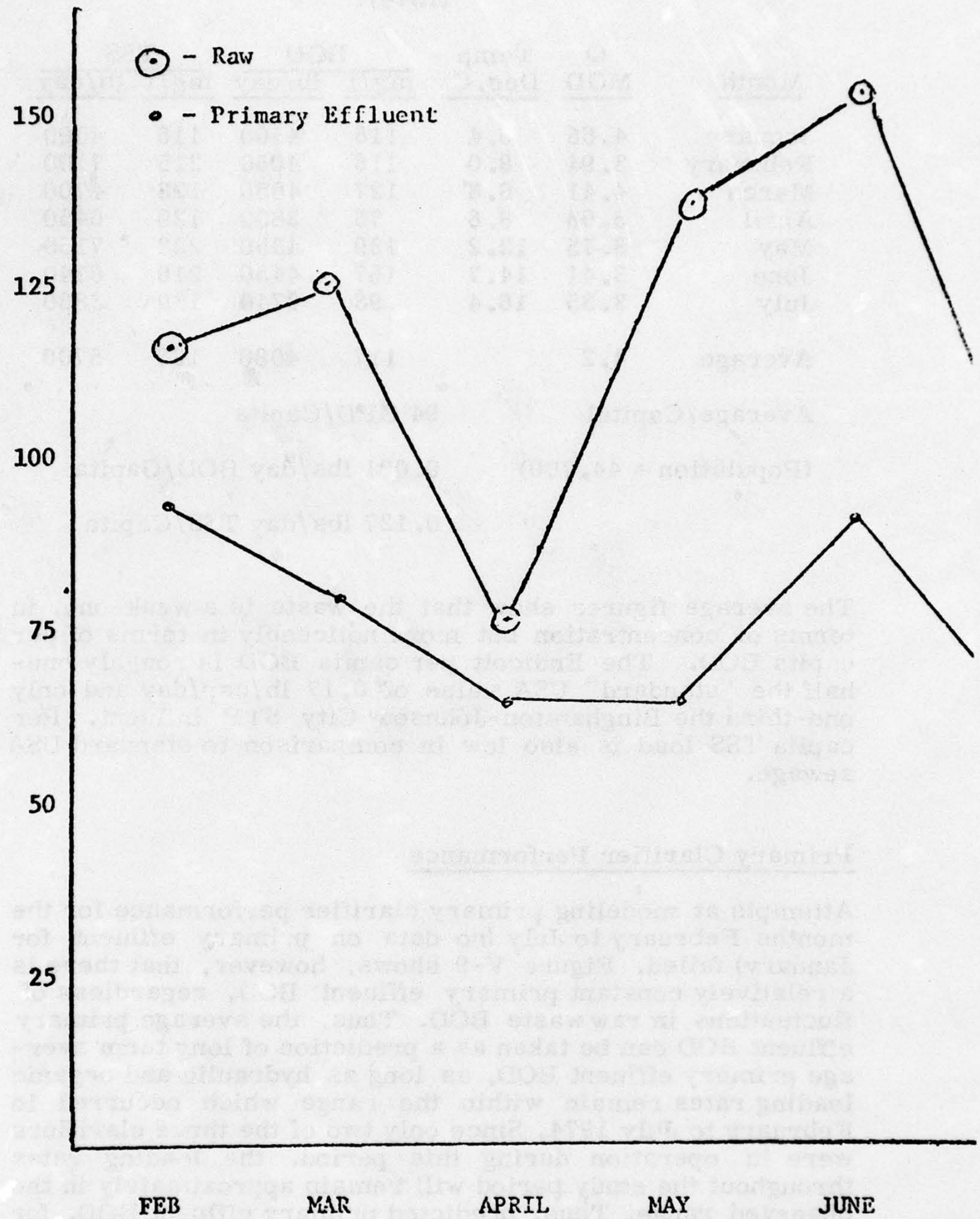
0.127 lbs/day TSS/Capita

The average figures show that the waste is a weak one, in terms of concentration but more noticeably in terms of per capita BOD. The Endicott per capita BOD is roughly one-half the "standard" USA value of 0.17 lb/cap/day and only one-third the Binghamton-Johnson City STP influent. Per capita TSS load is also low in comparison to standard USA sewage.

Primary Clarifier Performance

Attempts at modeling primary clarifier performance for the months February to July (no data on primary effluent for January) failed. Figure V-9 shows, however, that there is a relatively constant primary effluent BOD, regardless of fluctuations in raw waste BOD. Thus, the average primary effluent BOD can be taken as a prediction of long term average primary effluent BOD, as long as hydraulic and organic loading rates remain within the range which occurred in February to July 1974. Since only two of the three clarifiers were in operation during this period, the loading rates throughout the study period will remain approximately in the observed range. Thus, predicted primary effluent BOD, for an influent average of about 118 mg/l, was 76 mg/l, or a removal of 35 percent. This removal figure agrees well with normal primary clarifier performance on domestic wastes.

FIGURE V-9
ENDICOTT STP PRIMARY CLARIFIER
PERFORMANCE



Secondary Treatment Performance (Trickling Filter Plus Secondary Clarifier)

Secondary facilities at Endicott consist of two trickling filters and two clarifiers with characteristics shown in Table V-9.

TABLE V-9

ENDICOTT STP SECONDARY PROCESS CHARACTERISTICS

Trickling Filters (2)	Each	Total
Depth, Feet	6	
Area: SF	11,300	22,600
Acres	0.259	0.518
Volume, 1000 CF	68	136
Diameter, Feet	120	---
Secondary Clarifiers (2)		
Diameter, Feet	80	---
Area: SF	5,024	10,048

Recirculation is provided for unsettled and settled trickling filter effluent. During the period under analysis, only one filter was in operation, since the plant is presently handling about one-half of the rated capacity of 7.67 mgd influent flow.

Several correlations of secondary performance were tried, with some success; however, the BOD removal performance was poor. Figure V-10 indicates a correlation, but the scatter is quite high. This figure shows the average BOD removal in the secondary system was only about 40 percent during this period. Overall STP BOD removal was 61 percent, with an average effluent BOD of 46 mg/l.

Because of the relatively poor performance of the Endicott secondary system, several comparative analyses were made. The first of these is shown on Figure V-11 which compares the 1974 performance of the Endicott secondary to the design curve given by 10 State Standards (1971 edition). Only rarely did the Endicott plant match this design performance. The design curve should represent average performance, with the expectation that performance would be superior to that average about one-half the time.

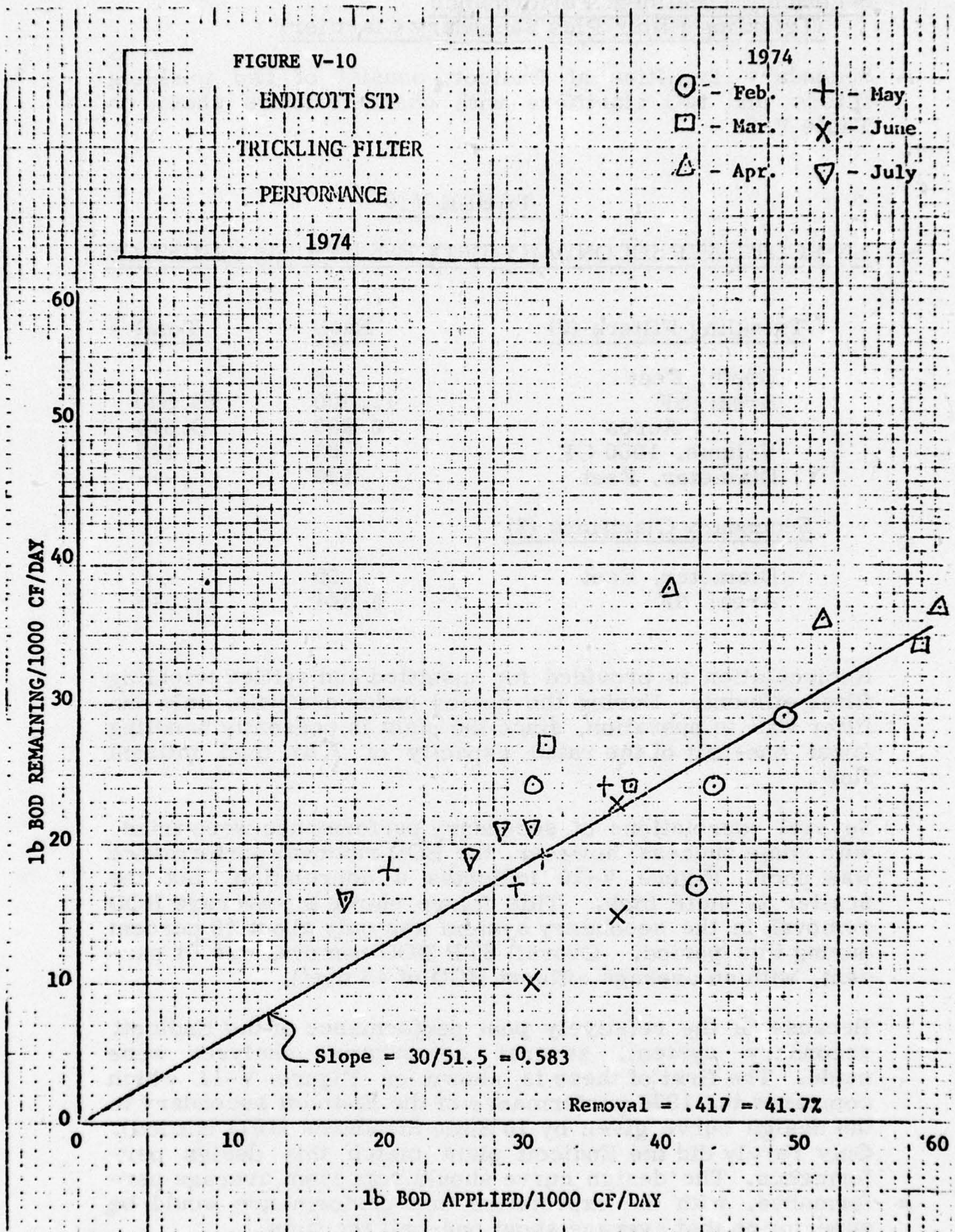
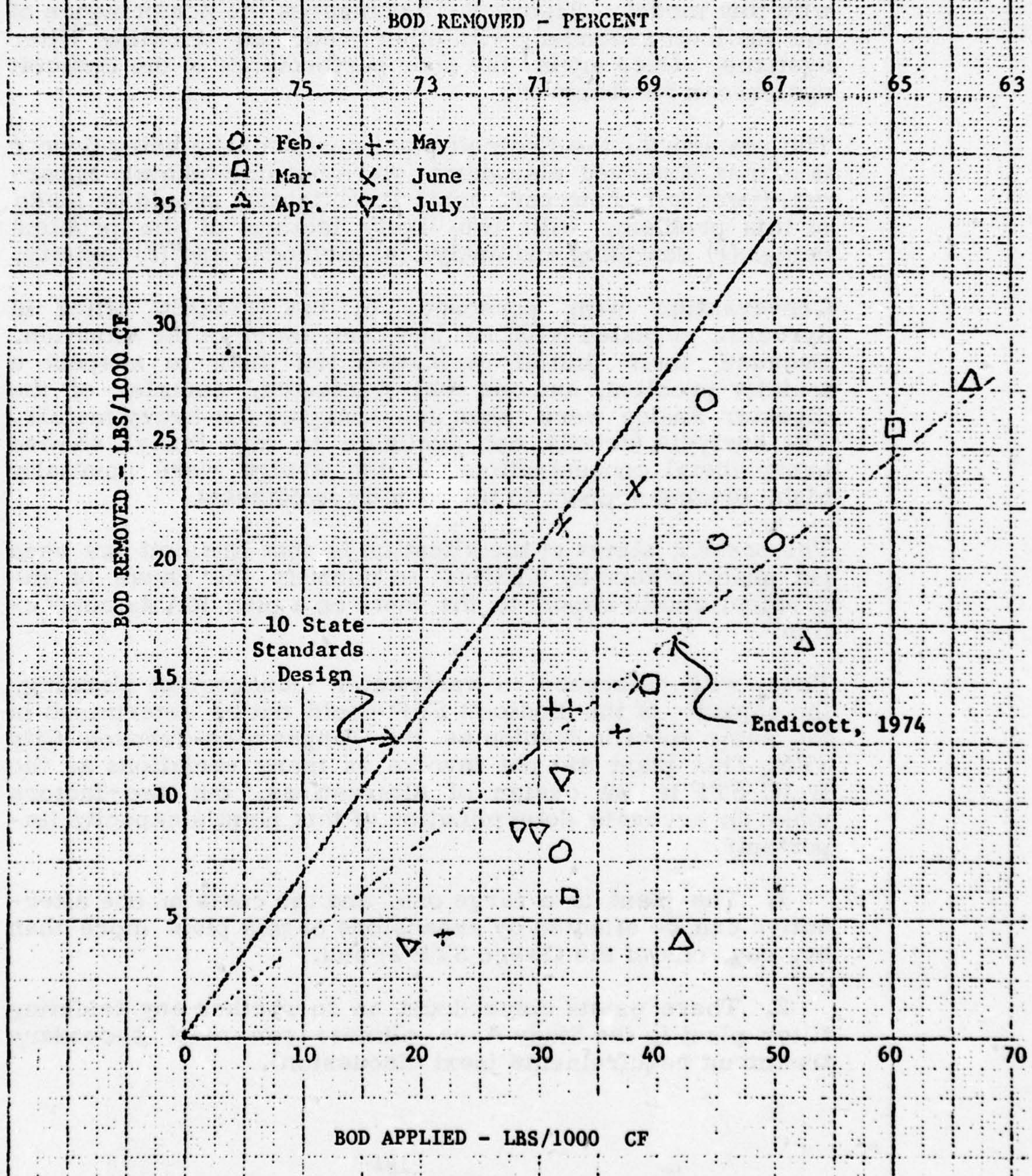


FIGURE V-10
ENDICOTT STP
TRICKLING FILTER
PERFORMANCE
1974
1974
- Feb.
- Mar.
- Apr.
- May
- June
- July
lb BOD REMAINING/1000 CF/DAY
lb BOD APPLIED/1000 CF/DAY
0
10
20
30
40
50
60
Slope = 30/51.5 = 0.583
Removal = .417 = 41.7%

FIGURE V-11
Endicott STP
BOD REMOVAL

TRICKLING FILTER UNITS

INCLUDING FINAL SETTLING TANKS



Performance was also analyzed by a model (O'Conner and Eckenfelder, 1966), which states that

BOD remaining = f [D(0.67)/Q(0.5)]

Where: D = Filter depth, feet

Q = Hydraulic loading, 10(6) gal/acre/day.

Figure V-12 shows there is only a weak correlation of data with this model. Figure V-13 compares the performance of the Endicott secondary with other reported trickling filter systems. Once again, the poor performance of the Endicott system can be observed.

The reasons for the Endicott plant performance being poorer than that observed elsewhere, were not immediately apparent. It was felt, however, some investigation should be made of this problem, with two likely factors of the problem being: (1) sampling and analytical methods, and (2) toxicity.

Conversation with personnel of the NYSDEC office in Syracuse revealed NYSDEC had been aware of the situation. Because their initial evaluation led them to suspect a toxicity problem, and one that fluctuated, samples of the digester sludge were taken and analyzed for heavy metals. The analysis (details not available to this Study) showed heavy metal concentrations in the sludge were unusually high, lending support to the toxicity hypothesis.

The present status of the situation is that Endicott has been put under orders by NYSDEC to identify the cause of the problem, and to report to the State on a plan for its correction.

Thus, with reference to wastewater management planning, the capacity of the Endicott STP could not be determined in the same specific fashion as the Binghamton-Johnson City STP. This plant did not involve as many variations as the B-JC STP in the design of alternatives, but two factors make an accurate determination of this plant's capacity important:

1. The plant is a large one, and the costs of one alternative can be affected by expansions of this plant more than by, say, one of the Owego STP's; and,

2. There exists some doubt as to whether any trickling filter plant in the Study Area climate can meet secondary treatment requirements (next discussion).

FIGURE V-12

BOD REMOVAL

AS A FUNCTION OF DEPTH AND FLOW
EDICOTT STP

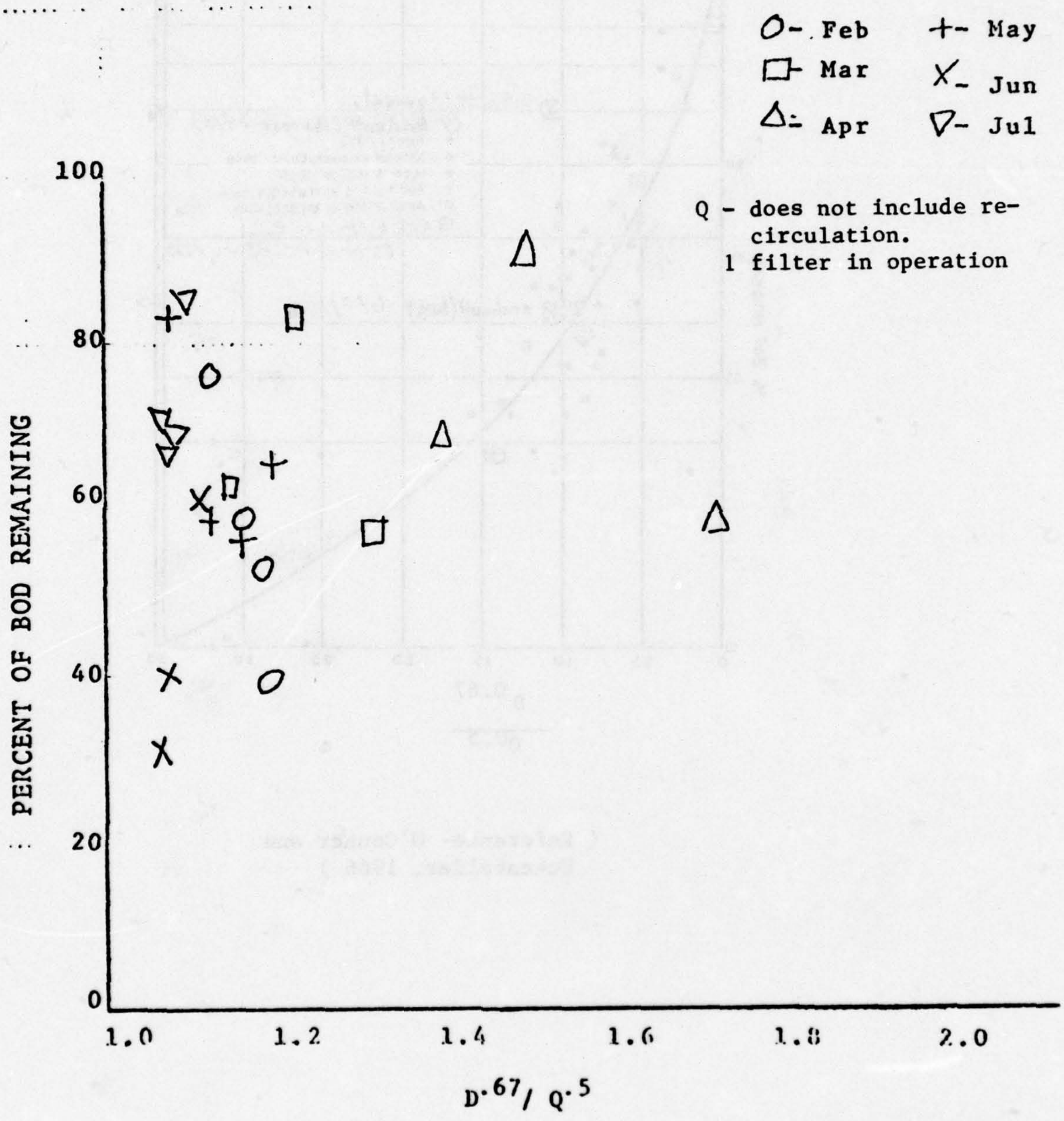
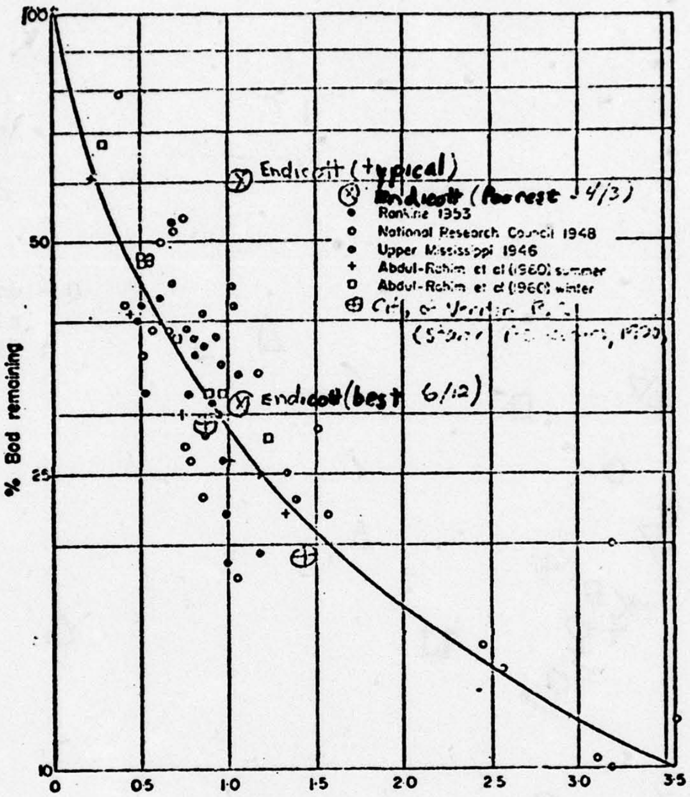


FIGURE V-13
CORRELATION OF TRICKLING
FILTER PERFORMANCE DATA
Endicott STP



$$\frac{D^{0.67}}{Q^{0.5}}$$

(Reference- O'Conner and
Eckenfelder, 1966)

Comparison of Endicott STP Performance to Definition of Secondary Treatment

Secondary treatment (for BOD) is defined by EPA as an effluent (30 day average basis), of 30 mg/l or less, and in addition, a minimum of 85 percent BOD removal, based on raw waste. For the months January to July 1974, Endicott's STP did not meet this standard as the effluent BOD averaged 46 mg/l, and the BOD removal was 61 percent.

It was pertinent to examine whether the plant (which is designed as a high rate trickling filter), could meet the definition of secondary treatment, even if STP performance was improved. For Endicott with its weak influent (some of its "weakness" could be due to toxicity in the BOD bottle), 85 percent removal would produce an effluent of 17.5 mg/l, so the 85 percent removal requirement, not the effluent limit of 30 mg/l, would be the controlling factor.

If the primary clarifiers continue to remove 35 percent of the influent BOD, required removal by the secondary is:

$$100 \times \frac{0.85 - 0.35}{1.0 - 0.35} = 100 \times \frac{0.5}{0.65} = 77\%$$

Referring back to Figure V-11, the 10 State Standards Design Curve, it can be seen that 77 percent removal can be approached only at very low organic loading rates, and it is probably impossible to attain. This point is discussed in a report on secondary treatment processes for the Binghamton area (Clinton Bogert, 1967), which states that "...the majority of high rate trickling filters will not routinely meet the Secondary Treatment Standard."

The importance of this question was underlined by NYSDEC's delay in approving the plans for Owego Village upgrading to secondary by trickling filter, until further documentation was submitted that the plant would meet the criteria for secondary treatment. The current plans for the Owego Village STP now use an activated sludge process.

Conclusions

The following statements summarize the dilemma posed by the Endicott STP to the present Study:

1. Doubts have been expressed by NYSDEC and EPA personnel as to the ability of trickling filter plants in the Study Area climate to meet secondary treatment criteria in the winter months;

2. The Endicott STP performance could not be analyzed, apparently due to a toxic substance, or substances, in its influent; and,

3. The design of the Endicott plant is for 85 percent BOD removal, i.e., secondary treatment.

If the Endicott STP toxicity problem is solved, it is likely that, even if it does not meet 85 percent BOD removal, it will meet on the order of 82 percent removal at design flows, and the difference in these two treatment levels was found to be insignificant in terms of resulting river DO. Since the objective of all alternatives which include secondary treatment at Endicott is maintenance of a given level of DO in the river, it was felt that the analysis of alternatives would be unrealistically distorted by including the additional capital works to improve from 82 percent to 85 percent removal.

Therefore, the existing Endicott treatment plant was assumed to have the rated design capacity of 7.67 mgd for secondary treatment. In alternatives where expansion of secondary capacity was required, parameters used in the design of the existing secondary units were carried through.

EAST AND WEST OWEGO AND OWEGO VILLAGE TREATMENT PLANTS

Existing Situation and Design

Besides the B-JC and Endicott STP's discussed in detail above, there are two existing secondary STP's in the Town of Owego: No. 2 (East Owego) and No. 1 (West Owego). In addition, the existing primary plant at Owego Village is scheduled for upgrading to secondary by 1977, and was included in the Baseline Profile as a secondary treatment plant.

The East Owego STP performance was not analyzed in detail, since it is presently well underloaded: 1973 flow was 0.4 mgd in comparison to a design flow of 2.0 mgd. The average of 1973 performance is shown in Table V-10.

TABLE V-10

OWEGO STP #2 (East Owego) PERFORMANCE
(1973)

	Influent mg/l	Effluent mg/l	% Removal
BOD	145	10	93
SS	135	17	88

Thus, this activated sludge plant is achieving treatment well above secondary, and it can be anticipated that it will maintain secondary levels until it reaches its design capacity. Design of expansion of this plant used the same unit design criteria as was used in the original design.

At the West Owego STP (trickling filter), average 1973 performance is shown in Table V-11.

TABLE V-11

OWEGO STP #1 (West Owego) PERFORMANCE
(1973)

	Influent mg/l	Effluent mg/l	% Removal
BOD	156	28	82
SS	182	38	79

Thus, this plant does not quite achieve 85 percent BOD removal, although its effluent BOD is less than 30 mg/l. SS removals are less than 85 percent, and effluent is greater than the 30 mg/l standard. The nominal capacity of the trickling filter and secondary clarifier at this plant is 1.0 mgd, but, as at Endicott, it is clear that the plant cannot achieve 85 percent BOD removal at that rate. The removal of SS at the plant is even poorer than the BOD removal.

As at Endicott, it was assumed, for the purposes of this Study, the trickling filters can achieve secondary treatment at their nominal capacity of 1.0 mgd. However, since secondary clarification (of SS) is poor, which probably also accounts for the poor BOD removal, all alternatives requiring expansion of this plant beyond 0.5 mgd included a new secondary clarifier, designed at the same loading rate as the existing unit of 0.5 mgd.

It was anticipated the Owego Village plant would be upgraded by installation of a trickling filter secondary, rated at 1.0 mgd capacity, i.e., equal to the rating of the existing primary plant. Designs for the Baseline Profile and all the alternatives were based on this assumption. Ten State Standards were used for design criteria.

It was recently learned, however, this plant will probably be upgraded as an activated sludge plant. Although O&M costs will thus be slightly higher at this plant than assumed in this Study, the comparative evaluation of alternatives would not be significantly affected by this change.

Effluent Loads

Effluent loads from theTioga County treatment plants were computed for all alternatives as follows:

1. Influent loading was assumed to equal "standard" sewage:

$$BOD = 0.17 \text{ lb/capita/day}$$

$$SS = 0.17 \text{ lb/capita/day}$$

2. For all alternatives (except the Baseline), expansions to meet secondary treatment standards were included and it was assumed:

Type of Plant	BOD Removal	SS Removal
Trickling Filter	85	85
Activated Sludge	90	90

3. For the Baseline Condition, in which flows were allowed to exceed design capacity, an analysis was made of the expected deterioration in effluent BOD quality, based on 10 State Standards curves for primary clarifier and trickling filter performance. This analysis assumed that overall removal would be 85 percent at design flow rate, i.e., the actual 10 State Standards curve was adjusted so that secondary performance would be achieved at design flow. The trend of deteriorating performance was taken after this adjustment. The result of this analysis is shown on Figure V-14. For the East Owego activated sludge plant, a curve parallel to the computed trickling filter curve, but achieving 90 percent BOD removal at design flow, was drawn. Effluent SS was assumed equal to effluent BOD in all cases.

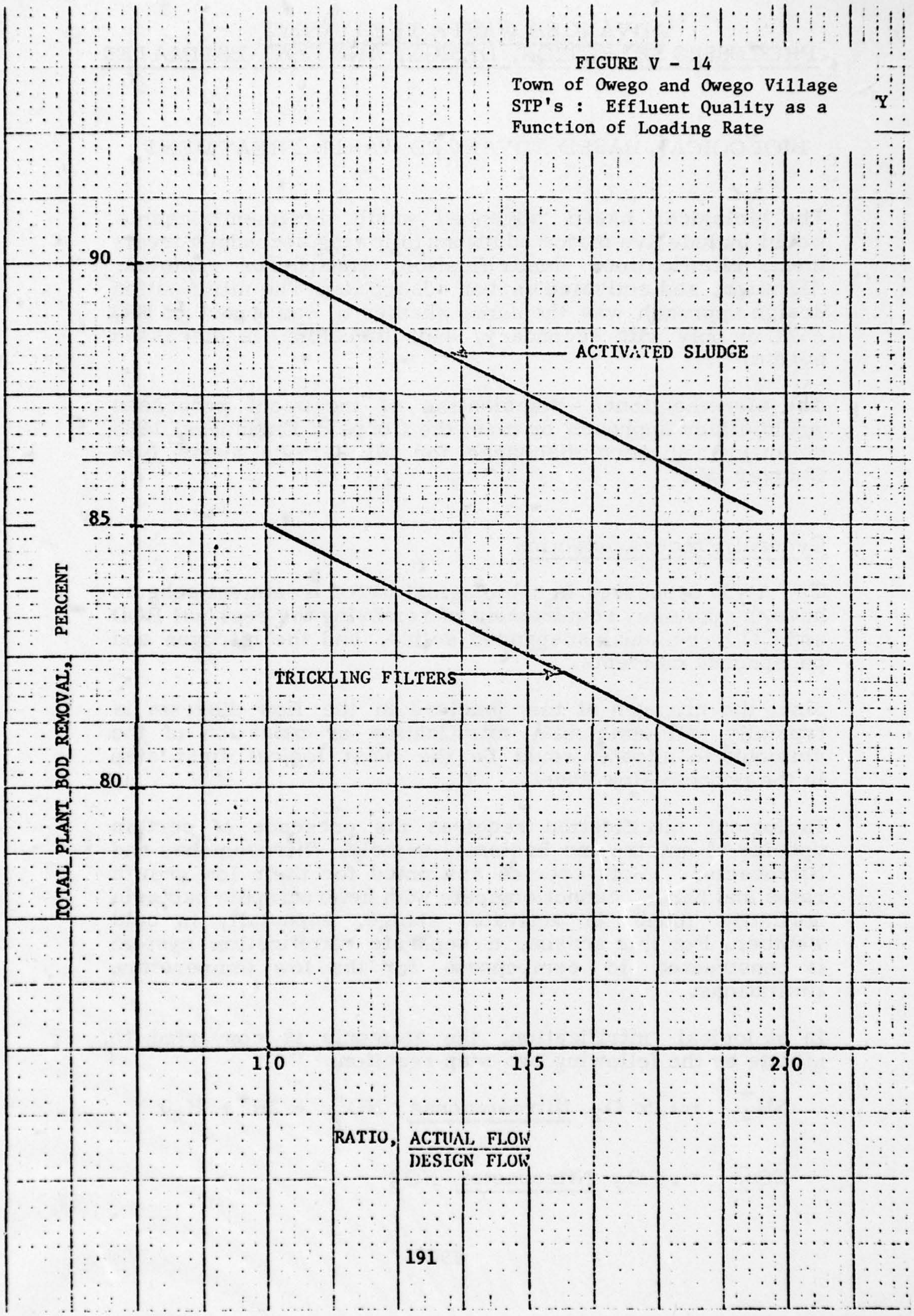


FIGURE V - 14
Town of Owego and Owego Village
STP's : Effluent Quality as a
Function of Loading Rate
ACTIVATED SLUDGE
TRICKLING FILTERS
90
85
80
TOTAL PLANT BOD REMOVAL, PERCENT
1.0
1.5
2.0
RATIO, ACTUAL FLOW / DESIGN FLOW

BIOLOGICAL BASED ADVANCED WASTE TREATMENT

The biological based advanced waste treatment scheme would include five major additions following secondary treatment: nitrification, denitrification, phosphorus removal, filtration, and activated carbon adsorption. The nitrification design approach was the same whether it was a part of BIO AWT or only with secondary, and, therefore, is discussed herein.

The aforementioned combination of treatment processes would either approach or meet the Corps of Engineers' 1985 definition of zero discharge for all service areas (see Chapter IV).

General Process Selection

The AWT processes in a biological based treatment scheme have the primary requirement of removing the residual BOD and COD, residual suspended solids, and the nitrogen and phosphorus nutrients.

Since the nitrogen at this juncture in the flow diagram is typically all ammonia, nitrification or oxidation of the ammonia to nitrate would be the most logical first step in the process flow sheet.

Biological nitrification requires the presence of certain species of autotrophic bacteria, namely, Nitrosomonas and Nitrobacter. Both species are noted for their low growth rates and hence, cannot compete with heterotrophic bacteria normally found in activated sludge, especially in cold weather. For this reason, a separate nitrification system is constructed to compensate for the low temperature limitations.

In biological nitrification, the ammonia is converted to nitrate by the following two-step reaction:

$$NH_4^+ \quad + 1.5\ O_2 \ \underline{\text{Nitrosomonas}} \ NO_2^- \quad + 2H^+ + H_2O$$

$$NO_2^- \quad = .5\ O_2 \ \underline{\text{Nitrobacter}} \ NO_3^-$$

It should be noted from the above equations that 1 mg/l of NH_3 -N oxidized to NO_3 -N, requires 4.57 mg/l of oxygen and neutralizes 7 mg/l of alkalinity. A minimum of 2 mg/l of DO should be supplied to maintain nitrification.

Following nitrification, the next step in AWT would be nitrate removal or denitrification. Biological denitrification requires the presence of nitrate or nitrite nitrogen, anaerobic conditions and a sufficient quantity of organic carbon. In denitrification, facultative heterotrophs utilize the nitrate as the electron acceptor for the oxidation of organic carbon. *This reaction is as follows:*

$$NO_3^- + 1/3\ CH_3\ OH\ ---\ NO_2^- + 1/3\ CO_2 + 2/3\ H_2O$$

$$NO_2^- + 1/2\ CH_3\ OH\ ---\ 1/2\ N_2 + 1/2\ CO_2 + 1/2\ H_2O + OH^-$$

Due to the low input of organic carbon at this point, a supplementary organic carbon source is required. Methanol is usually chosen because it has a low cost per volume of wastewater treated, and because bacteria utilize methanol (MeOH) primarily for oxidation and not for synthesis. Therefore, an MeOH based denitrification system does not yield high quantities of sludge.

Since denitrification produces excess quantities of nitrogen and carbon dioxide gases, a post aeration system would be required to drive off these gases and to make solids settling easier.

Because the post aeration facility needs a high degree of mixing, it was concluded the system could also be utilized as the coagulation-flocculation chamber for alum addition.

The alum would react with the phosphorus to form an insoluble settleable floc which could be removed in the denitrification settling chamber.

Following denitrification, a certain amount of suspended solids will remain in the wastewater stream. These can be removed utilizing a multimedia filter. The filter will also remove the residual organic phosphorus associated with the suspended solids. It would also be desirable to filter at this point to avoid clogging of the activated carbon columns.

Activated carbon will remove approximately 50 percent of the remaining BOD-COD in the wastewater. Higher removals can be obtained only at an increased economic investment.

After chlorination for disinfection purposes, the wastewater should meet or approach the zero discharge goal (as interpreted by the Corps of Engineers) for the major constituents of known concentration in the Study Area wastewaters.

Design Criteria and Detailed Considerations

Nitrification.

The Cornell University report entitled "Biokinetic Approach to Optimal Design and Control of Nitrifying Activated Sludge Systems" was utilized in determining some of the design criteria. Their study involved a comparison of nitrification in a one sludge or two sludge system.

Utilizing their sludge settling data for the separate nitrification system at cold temperatures, the solids flux rate was calculated as follows:

$$Gg = Vi\ Xi$$

Where: Gg = solids flux, lb/day/SF

Vi = settling velocity, fpm

Xi = solids concentration, mg/l

When the initial solids concentration was plotted against the solids flux rate, a batch flux curve was obtained. This plot yielded the underflow solids concentration and the solids flux rate for this particular system. These values were: 9,000 mg/l and 30 lb/d/SF, respectively. The above values in turn yielded the overflow rate for the clarifier of 740 gpd/SF.

On the basis of these data, the underflow concentration and overflow rate utilized in the design of the nitrification system were: 9,000 mg/l and 700 gpd/SF, respectively. Additionally, 72 percent volatile solids was used in the determination of MLSS concentration. In all cases, it was assumed the separate nitrification system could maintain an MLVSS concentration of 2,000 mg/l (typical value). This resulted in a MLSS figure of 2,780 mg/l.

For alternatives requiring nitrification, the influent ammonia concentration for the Binghamton-Johnson City STP was based on actual data for the treatment plant. In the remaining plants the ammonia nitrogen concentration was determined on the basis of an assumed value of 0.03 lb/cap/day. At 10 percent winter removal in the secondary

treatment system, this would be equal to a 0.027 lb/cap/day load to the nitrification system. The 0.027 lb/cap/day is a somewhat conservative value for the plants which utilize activated sludge for secondary treatment. However, this value was used for simplicity of design.

The EPA Technology Transfer Manual, "Nitrification and Denitrification Facilities," was utilized in calculating the sizes of the aeration tanks and clarifiers. Knowing the minimum temperature conditions (12 degrees C) and the MLVSS concentration (2000 mg/l) permitted the computation of the loading rate in lb NH3 -N/day/1000 CF from a graph. The value of 12.5 lb/day/1000 CF, in conjunction with the influent value, yielded the aeration tank required volume. However, since nitrification is strongly pH dependent, this size had to be corrected for pH differences from the optimum value of 8.0. (The alternative approach would be to raise or lower the pH to optimum conditions by chemical addition. In the Study Area, the influent pH is approximately 7.0, which would require significant addition of caustic.) To correct for the size of the tank, two schools of thought exist as to the exact dependency of nitrification on pH. The EPA data illustrate a more pronounced effect than the data of Downing, et. al., (1964). Utilizing both correction factors resulted in a range of sizes. Therefore, it was estimated a value halfway between the two would be sufficient. The loading rate was then calculated to be 7.6 lb NH_3 -N/d/1000 CF.

The sizes of the nitrification system for Stage II-2 flows for the various plants are given in Table V-12. Two of the plants, East Owego and West Owego, have a second expansion included. On the basis of an economic analysis, it was determined more benefits accrued from expanding when the design capacity is reached rather than design a system to handle the capacity for the entire planning period.

The capital costs for nitrification are also included in Table V-12. These costs were derived from the actual tank sizes required. However, since flows, and, consequently, tank sizes were changed in subsequent iterations, a curve was prepared from the costs in the table, on a flow basis as shown in Figure V-15. This cost curve was used to estimate the capital costs for nitrification in Stage III plans.

Denitrification.

The major reference utilized in the design of the denitrification system was the EPA Technology Transfer Manual,

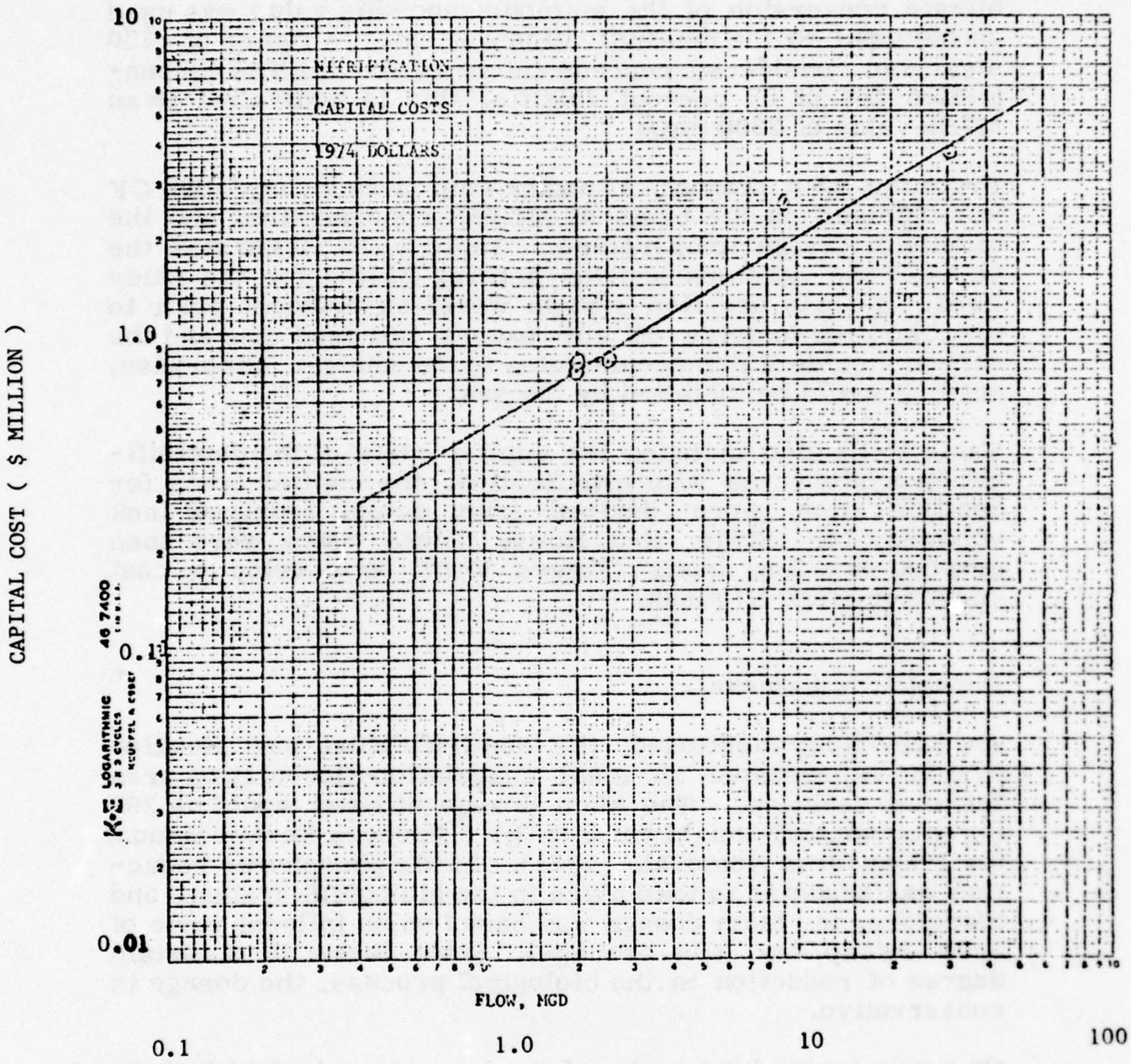
TABLE V-12

NITRIFICATION TANK SIZES

Plant	Design Flow[1] (MGD)	Influent NH_3-N lb/day	Aeration Tank Size (1000 CF)	Detention Time (hrs)	Clarifier Surface Area (ft^2)	Additional Flow[1] (MGD)	Additional Aeration (1000 CF)	Additional Clarifier (ft^2)	Capital Cost[2] ($\$10^6$)	Additional Capital Cost (Expansion ($\$10^6$)
Binghamton-Johnson City: Alternative 3	20.8	2640	366	3.2	29,800	–	–	–	3.67	–
Endicott	9.2	2050	270	5.2	13,100	–	–	–	2.54	–
Chenango Valley	2.5	675	89	6.4	3,580	–	–	–	0.770	–
E. Owego	2.0	445	58	5.2	2,860	1	29	1430	0.792	0.415
W. Owego	2.0	367	48	4.3	2,860	0.5	12	715	0.750	0.255
Binghamton-Johnson City: Alternatives 1, 2, 4	25.3	2790	366	2.6	36,200	–	–	–	–	–

[1] For Stage II-2 Flows
[2] ENR 2135

FIGURE V-15
Nirtification Capital Costs
(1974 Dollars)



"Nitrification and Denitrification Facilities." According to this manual, denitrification organisms compare closely to activated sludge floc in settling and compaction characteristics. On this basis, an underflow concentration of 10,000 mg/l and an overflow rate of 800 gpd/SF was utilized in the design, as these values are typical in conventional activated sludge systems.

The nitrification system was designed to yield 90 percent nitrate conversion of the ammonia and this value was used as the input to the denitrification system. A figure of 1500 mg/l was considered representative of the MLVSS concentration and at 65 percent volatile, this is equivalent to an MLSS value of 2300 mg/l.

From the EPA manual, a loading rate of 25 lb/day/1000 CF was obtained on the basis of MLVSS concentration and the minimum design temperature. Table V-13 illustrates the sizes of the reactors and the settling basins for the Study Area. A flash aeration system would be provided prior to the clarifier to drive off the excess nitrogen gas and the carbon dioxide which accumulates in the sludge. Otherwise, settling would be inhibited by the gas.

Table V-13 also includes the capital costs of the denitrification system. As with nitrification, the capital costs for denitrification were derived from actual required tank volumes for Stage II-2 flows. These costs were then graphed on a flow basis (Figure V-16) to develop a cost curve for Stage III plans.

Phosphorus Removal.

As previously mentioned, the denitrification post aeration system represented an ideal coagulation-flocculation area for alum addition. The alum dosage utilized would be 200 mg/l or an Al:P weight ratio of 2:1 ("Process Design Manual for Phosphorus Removal," 1971). Little phosphorus reduction was assumed to take place in the biological system, and therefore, the alum dosage was based on an influent value of 0.01 lb/cap/day (US average). Since there is a certain degree of reduction in the biological process, the dosage is conservative.

On a per capita basis, the alum dose is equivalent to 0.22 lb/cap/day or 80 lb/cap/year. The sludge produced from the three biological systems in addition to that produced by the chemical addition would be somewhat greater than that produced in typical primary and secondary treatment. The

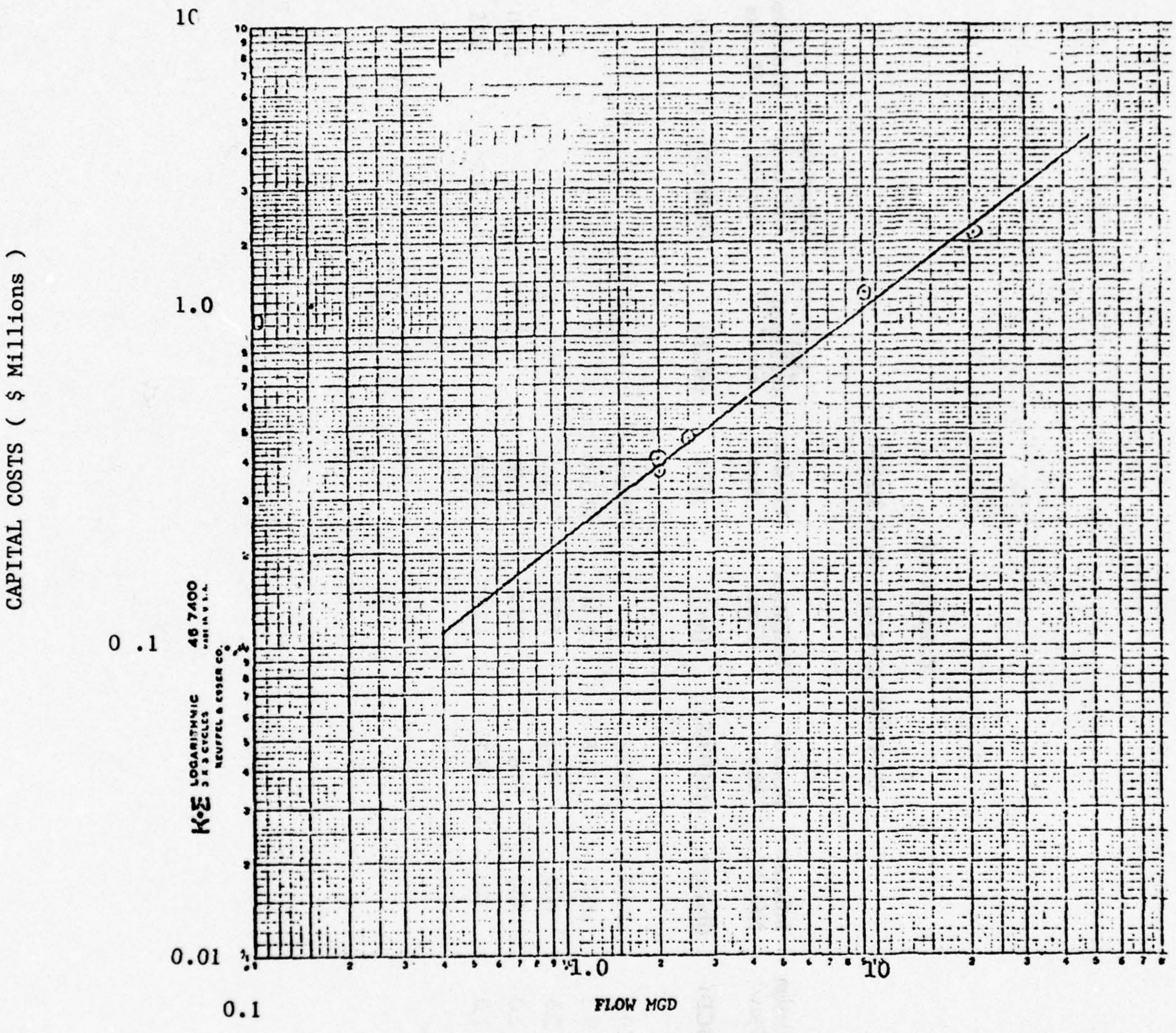
TABLE V-13

DENITRIFICATION TANK SIZES

Plant	Design Flow[1] (MGD)	Influent NO_3-N (lb/day)	Reactor Size (1000 CF)	Detention Time (hrs)	Clarifier Size (SF)	Next Additional Flow[1] (MGD)	Expansion Additional Reactor (1000 CF)	Additional Clarifier (SF)	Capital Cost[2] (10^6)	Additional Capital Cost[2] (Expansion) (10^6)
Binghamton-Johnson City	20.8	2637	115.8	1	26,000	–	–	–	2.13	–
Endicott	9.2	1845	73.8	1.44	11,500	–	–	–	1.31	–
Chenango Valley	2.5	608	24.3	1.75	3,125	–	–	–	0.460	–
E. Owego	2.0	400	16	1.44	2,500	1	8	1250	0.408	0.185
W. Owego	2.0	330	13.2	1.19	2,500	0.5	3.3	625	0.365	0.145

[1] Stage II-2 Flows
[2] ENR 2135

FIGURE V-16
DENITRIFICATION CAPITAL
COSTS (1974 Dollars)



total chemical sludge produced would be 0.0715 lb/cap/day (refer to P/C AWT discussion for equations).

The capital cost for phosphorus removal would be the same as that required for the P/C AWT alternative. For the cost curve utilized for this process, refer to the discussion under P/C AWT.

Filtration.

Multi-media filters would be employed to remove the residual suspended matter and the residual organic phosphorus. The design values utilized were: 5 gpm/SF throughput rate and a backwash volume of 3 to 5 percent. The effluent from the filter would be utilized for backwashing and this in turn would be recycled back to the primary clarifier for treatment. The backwash velocity would be approximately 15 gpm/SF.

The capital cost curve for filtration is presented in the P/C AWT discussion. There is no difference in construction cost for the filtration process in the two types of AWT systems.

Activated Carbon Adsorption.

The activated carbon adsorption columns would be necessary to remove the remaining BOD-COD. Without the benefit of pilot plant data, design criteria was gathered from literature sources. Since the biological based AWT scheme is similar to that employed at the South Lake Tahoe treatment plant, their design criteria were used in sizing the activated carbon system (for the biological based AWT alternative). On this assumption, the carbon dose would be 250 lb/MG or 30 mg/l with a contact time of 15 minutes. Carbon attrition amounts to 7.5 percent making regeneration economically desireable. The carbon columns should be either packed or expanded bed upflow columns in parallel with two trains. In order to obtain the highest efficiency from each column, counter-current flow should be applied, i.e., spent carbon is withdrawn from the bottom of the column and fresh carbon is placed in the top. This enables all of the adsorptive capacity of the carbon to be utilized before regeneration. Regeneration with multiple hearth furnaces requires 1 SF/100 lbs carbon regenerated per 24 hours. The columns should have an L:D ratio of greater than 2.0 and a depth greater than 10 feet.

The cost curve for carbon adsorption is presented in the P/C AWT discussion. The curve was derived from the 1969 South Lake Tahoe data and scaled up to 1974 dollars. Eiler's data (P/C AWT) were utilized for the economy of scale. The curve used for the activated carbon process for the BIO based system was drawn parallel to it and through the data point for South Lake Tahoe.

PHYSICAL/CHEMICAL ADVANCED WASTE TREATMENT

Physical/chemical advanced waste treatment (P/C AWT) in this Study did not include any biological processes in order to be a true alternative to the biological treatment schemes. The components of the P/C AWT system also were designed to meet the Corps of Engineers' definition of zero discharge. The proposed system should meet all of the requirements with the possible exception of BOD, phosphorus, and sulfates; however, where targets were not met exactly, they will be approached by P/C AWT.

To determine the best pathway to the no discharge goal, the P/C system was designed to go directly from secondary to advanced waste treatment without the intermediate step of BPWTT. In a biological system, little advantage would arise from utilizing this route, since BWPTT (nitrification as defined in this Study) is a "stepping stone" to AWT. However, a physical-chemical scheme does not have the effluent quality of BPWTT at any point. In other words, any combination of physical-chemical treatment components will always yield an effluent quality either better than or worse than BPWTT.

General Process Selection

Four major sewage contaminants--TSS, BOD (COD), phosphorus and nitrogen--were the primary parameters of concern in the selection of a flow diagram.

The first step in a physical/chemical scheme is, typically, chemical coagulation followed by settling. This permits the removal of a considerable portion of the TSS and dissolved phosphorus as well as the BOD-COD associated with suspended matter. Little nitrogen removal occurs at this stage.

In order to remove the influent ammonia nitrogen three possible mechanisms exist. These are ammonia stripping, ion exchange and breakpoint chlorination. Ammonia stripping is severely hampered by cold weather which precluded its use in the Binghamton area. Ion exchange, although competitive on a cost basis with breakpoint chlorination, requires significant chemical addition and is a complicated operating system. Breakpoint chlorination has the disadvantage of requiring post activated carbon adsorption for removal of the chloramines. However, since activated carbon is necessary in a P/C AWT scheme in order to remove the BOD-COD, this disadvantage would be removed. Due to the simplicity in design and estimating costs, breakpoint chlorination was utilized as the nitrogen removal system.

Multimedia filtration would remove residual TSS, reduce the suspended phosphorus concentration, and would further reduce the BOD-COD. In order to avoid clogging problems in the activated carbon facility, it was also decided to filter prior to adsorption. In consideration of the above recommendation, filtration was positioned after breakpoint chlorination and prior to activated carbon.

Activated carbon is a physical/chemical unit process which reduces the BOD-COD to the desired level as well as removing the residual chloramines. Since activated carbon is normally employed as a tertiary treatment step following biological waste treatment, the carbon requirements to remove the significant portion of the BOD remaining at this juncture would be considerable. In light of this factor, the activated carbon step would be the most expensive. The cost is strongly dependent on the wastewater characteristics, especially in regard to organics. Industrial contributors could strongly influence the characteristics of influent organics. A pilot plant is normally conducted in order to calculate the exact carbon dosage requirement, but in lieu of this, a literature search in combination with the sewage characteristics and judgment, yielded approximate dosages.

<u>Design Criteria and Detailed Considerations</u>

The P/C AWT scheme assumed a biologically based secondary sewage treatment plant was already installed. Additionally, the preliminary treatment processes--grit removal, screening, pumping, sampling, metering--would be unaffected by the P/C AWT process.

Chemical Coagulation and Settling.

The primary clarifiers could be readily converted into flocculation clarifiers with new units supplied as needed. A rapid mix zone would be located prior to the tanks requiring a mixing time of one minute. In-line mixing would, therefore, probably be utilized. A flocculation zone with a detention time of 15 minutes, would be provided within the tank. The clarifier would operate at an overflow rate of 900 gpd/sf at average flow and 1400 gpd/sf at maximum flow.

The chemical utilized for clarification and phosphorus removal can be either lime, alum, or iron. Lime produces excessive quantities of sludge which cannot be anaerobically digested. However, lime has the advantage of recalcination, or reuse. This advantage of lime would be lost with its addition to primary clarifiers, due to the high organic content of the sludge and the consequent low concentration of CaCO3. These factors make recalcination economically unfeasible. Alum and iron are equally good coagulants and competitive in price, but iron imparts color to the effluent. In light of this, alum was chosen as the chemical to be utilized. Alum is difficult to dewater but can be anaerobically digested. Land disposal of alum sludge will not cause the phosphorus to leach out, and will also retain the organics.

The quantity of sludge produced from alum addition would be approximately equal to that produced from a secondary treatment plant. For 95 percent phosphorus removal, an Al:P weight ratio of 2:1 is required. This is equivalent to an alum dosage of 200 mg/l. The influent phosphorus concentration was unknown but has been assumed to be 0.01 lb/cap/day of phosphorus. The corresponding alum dosage is 0.22 lb/cap/day, or 80 lb alum/cap/yr. At a price of $80/ton, this would be equivalent to $3.20/cap/yr.

The alum sludge quantity was calculated from stoichiometric considerations. When alum is applied to wastewater, two precipitates result: $AlPO_4$ and $Al(OH)_3$. The quantities of sludge produced were calculated as follows:

1. $AlPO_4$ sludge = $\Delta P(3.94)(Q)(8.34)$ where ΔP equals mg/l phosphorus removed. At 95 percent phosphorus removal ΔP equals 0.0095 lb/cap/day and the $AlPO_4$ sludge is equal to 0.0374 lb/cap/day.

2. $Al(OH)_3$ sludge = $(Al - 0.87\,\Delta P)(2.89)$ where Al equals alum dosage expressed as aluminum. This is equivalent to 0.0341 lb/cap/day of sludge or a total chemical sludge quantity, including $AlPO_4$ of 0.0715 lb/cap/day.

The capital cost for phosphorus removal included the costs for chemical storage and feeding equipment. The cost curve was derived from the EPA Manual for phosphorus removal and scaled up to 1974 costs. This curve is presented in Figure V-17.

Breakpoint Chlorination.

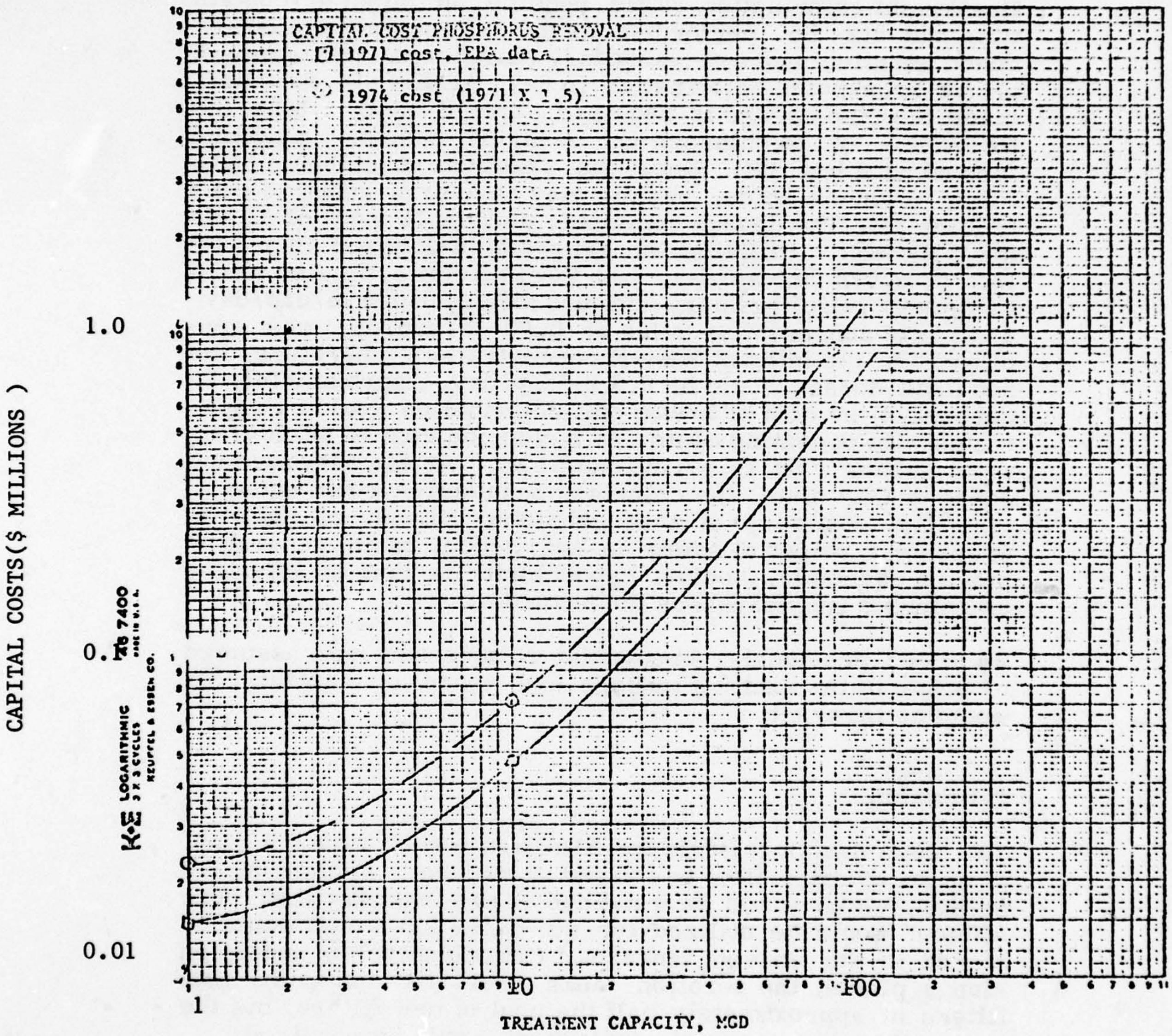
Breakpoint chlorination would require 30 minutes contact time and may be performed in the existing tankage (abandoned aeration tanks or secondary clarifiers). The optimum pH for breakpoint chlorination is 6-7. At this pH, the minimum chlorine dosage is required, and it also avoids the formation of trichloramine, a highly noxious compound formed at a low pH. Since the reaction of chlorine with water produces acids, caustic may have to be added to control the pH. The dosage commonly used is 10 mg/l of Cl per mg/l NH_3-N. Utilizing the Binghamton-Johnson City STP load of 0.028 lb/cap/day, and the Endicott estimated (from NH3/BOD ratio) primary effluent load of 0.025 lb/cap/day, a slightly conservative value of 0.03 lb/cap/day was employed for the influent NH_3-N concentration. Therefore, the required dosage would be 0.3 lb/cap/day. Approximately 80 percent of the cost of breakpoint chlorination would be associated with operating costs. At a chlorine cost of $0.067/lb, the resulting cost is $7.30/cap/year. At the Binghamton-Johnson City plant, the chlorine consumption would equal 15 tons/day. Delivery of this extensive amount of chlorine presents a potential safety hazard; hence, onsite generation may be advisable. Sodium hypochlorite is just as effective as chlorine and decreases the hazards.

As stated previously, breakpoint chlorination was assumed to occur in the existing tankage and hence no capital cost would be involved.

Filtration.

The basic design criteria for filtration were· flow = 5 gpm/sf on an average flow basis and a backwas . volume of 3-5 percent which would be returned to the primary tank. The effluent would be utilized for the backwash which would be stored in the unused tankage. Conceivably, in activated sludge plants, the aeration tanks could be converted into filters at approximately half the cost of new filters, but the cost estimating assumed new tankage would be required.

FIGURE V-17
PHOSPHORUS REMOVAL CAPITAL
COSTS



Eilers (1970) was the source for the construction cost for filtration. The costs were scaled up to 1974 dollars and are presented in Figure V-18.

Activated Carbon Adsorption.

In a P/C AWT scheme, the activated carbon process was designed to handle a higher BOD-COD load than a system following a bio-based scheme. The BOD-COD relationship at the Binghamton-Johnson City STP was utilized in the design of the activated carbon system. With an influent BOD of 200-205 mg/l and 31 percent soluble, the BOD load to the activated carbon would be 60-65 mg/l, or 0.09 lb/cap/cay.

Based on EPA information, the activated carbon (AC) to BOD dosage = 3.5:1 (or 1.7:1, AC:COD removal). This would remove essentially all BOD. In accordance with these dosages and an attrition rate of 7.5 perercent, Table V-14 outlines the carbon requirement and costs for various plants in the Study Area.

TABLE V-14 -- CARBON COSTS

STP	Raw BOD lb/cap/day	To AC @ 0.31 lb/cap/day	Carbon Dose lb/cap/day @ 3.5/1 AC/BOD	Required Carbon (Attrition @ 7.5%) lb/cap/day	lb/cap/yr
B-JC	0.29	0.09	0.315	0.0236	8.6
Endicott	0.13	0.04	0.145	0.0109	4.0
Others	0.17	0.053	0.185	0.0139	5.05

The design involves upflow columns in parallel with two trains. A loading of 5 gpm/sf was utilized with a contact time of 50 minutes. The contact time is a conservative estimate and was employed because of the lack of pilot plant data. The columns should have a length/diameter ratio of at least 2:1 and a depth of at least 10 feet. One column would always be down for regeneration purposes. Multiple hearth furnaces would be used for regeneration and these require approximately one square foot per 100 pounds of carbon regenerated each 24 hours.

The capital cost for carbon adsorption was obtained from Eilers (1970) and adjusted to 1974 dollars. The cost curve is given in Figure V-19.

FIGURE V-18
RAPID FILTRATION
CAPITAL COSTS

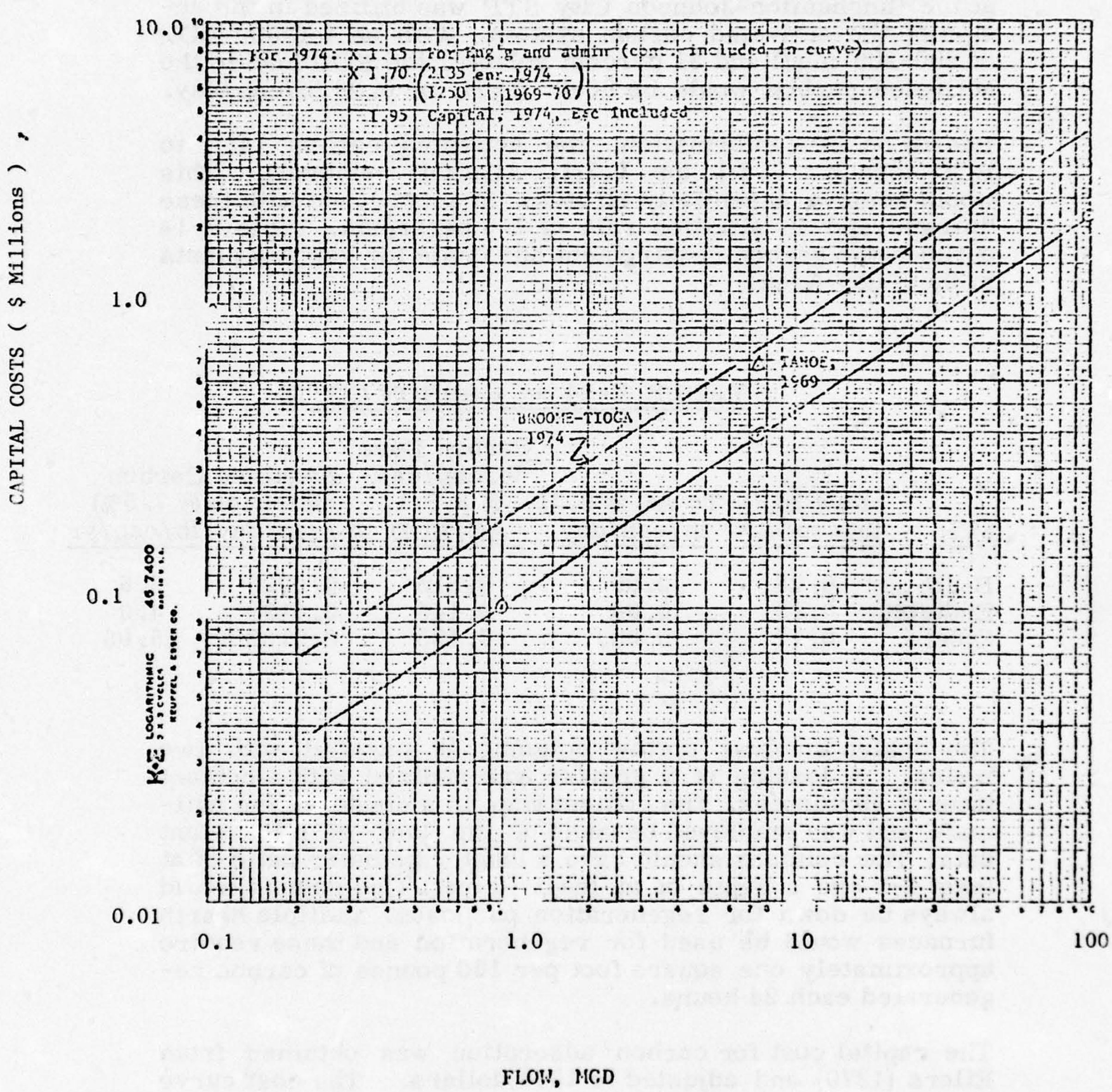
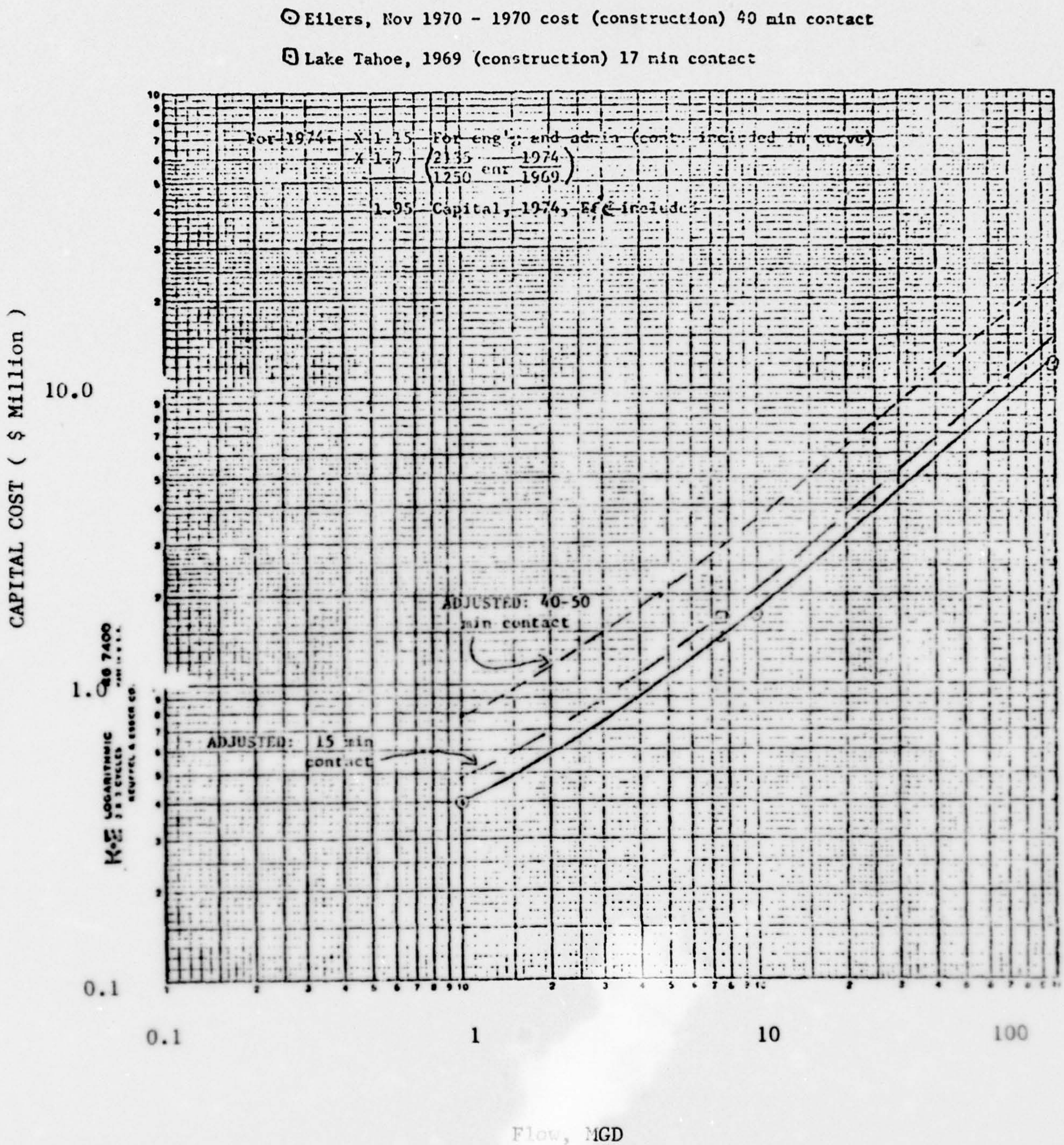


FIGURE V-19

ACTIVATED CARBON CAPITAL
COSTS



CHAPTER VI

SLUDGE MANAGEMENT

The alternatives for wastewater treatment developed in Stage II implicitly included the costs of sludge handling. Costs were obtained from curves which included sludge management or, in the case of secondary treatment, were based on actual operating experience in the Study Area. However, up to this point no analysis was performed on the various alternatives for sludge processing and disposal. The purpose of this Chapter is to document the selection of a specific sludge management scheme for each wastewater treatment level carried forward into Stage III.

An analysis of sludge management alternatives is important as 25 to 50 percent of the cost of conventional (primary and secondary) treatment can be charged to sludge processing and disposal. The fraction of treatment plant cost attributable to sludge disposal generally increases with increasing plant size. The method used to handle sludge from advanced waste treatment systems is critical to the successful and economic operation of the plant.

Sludge requires treatment to (1) reduce the volume of the material to facilitate subsequent handling and disposal; (2) decompose organic matter to stabilize the material and render it inoffensive; (3) destroy pathogens; and (4) utilize the gas produced by anaerobic digestion for heating and thereby improve the economy of the treatment.

EXISTING SLUDGE MANAGEMENT PRACTICES IN BROOME AND TIOGA COUNTIES

Table VI-1 summarizes approximate quantities and types of sludge generated and existing sludge handling practices for each of the sewage treatment plants. At present, the digested sludge generated from the seven sewage treatment

TABLE VI-1

EXISTING SLUDGE MANAGEMENT PRACTICES IN BROOME AND TIOGA COUNTIES

Name	Service Area	Sewered Population	Design Capacity mgd	Type of Treatment	Digested Sludge Produced Dry Solids T/Day	Method of Sludge Handling
Binghamton-Johnson City Joint STP	Binghamton City, Johnson City (V), Binghamton (T) S.D. 1,2,3,4,6&8, Kirkwood (T) S.D. 1, Vestal (T) S.D. 4,7,11&12, Dickinson (T) S.D. 2,3 & Stella District, Port Dickinson (T), Union (T) Westover S.D., SUNY	102,000	18.25	A/S or C/S and Chlorination	5.5	AD, MDW, VF, LF, or to public as soil conditioner
Vestal STP	Sewer Districts 1,5,6	7,500	1.0	PT and Chlorination	0.28+	AD, DB & to public as soil conditioner
Endicott STP	Endicott (V), Endwell, Park Manors, North Endicott, West Endicott, West Corners, Airport Heights	44,700	7.67	HR T/F and Chlorination	2.64	AD, MDW, VF, and LF
Owego (T) STP #1	Western Section of Town of Owego S.D. 1	600	0.5	T/F and Chlorination	0.19	AD, Liquid sludge for land application
Owego (T) STP #2	Eastern Section of Owego (T) S.D. 2, 3,6&8	6,500	2.0	A/S or C/S & Chlorination	0.42+	AD, Liquid sludge for land application

TABLE VI-1 (Continued)

EXISTING SLUDGE MANAGEMENT PRACTICES IN BROOME AND TIOGA COUNTIES

Name	Service Area	Sewered Population	Design Capacity mgd	Type of Treatment	Digested Sludge Produced Dry Solids T/Day	Method of Sludge Handling
Owego Village STP	Other than Valley View Service Area	3,600	0.62	PT and Chlorination	0.14+	AD, DB, & disposed to LF
Owego (V) Valley View STP	Small community in southern section of the Village	500		IT and Chlorination	0.02+	AD, DB, and to public as soil conditioner

List of Abbreviations:

AD – Anaerobic Digester
A/S – Activated Sludge Process
C/S – Contact Stabilization
DB – Drying Beds
IT – Imhoff Tank
LF – Land Fill
MDW – Mechanical Dewatering

PT – Primary Treatment
S.D. – Sewer District
STP – Sewage Treatment Plant
T/F – Trickling Filter (HR-High Rate)
VF – Vacuum Filter
(T) – Town
(V) – Village
+ – Estimates

plants in the urban area amounts to approximately 9.2 tons per day, based on dry solids. Assuming 20 percent solids concentration in the dewatered sludge, the amount of sludge to be hauled for disposal is approximately 46.5 tons per day or 1,400 cubic feet per day. With the use of trenches 10 feet deep, the total area required for landfill per year would be approximately 1.2 acres. Assuming land application at the rate of 10 tons of dry solids per acre per year, the land required per year would be approximately 340 acres. However, if the sludge were incinerated, the landfill area required for ash dumping in trenches 10 feet deep would be less than 0.23 acres per year.

BINGHAMTON-JOHNSON CITY JOINT SEWAGE TREATMENT PLANT

Dilute primary sludge is withdrawn continuously and distributed to three sludge thickeners. Excess activated sludge is withdrawn as needed. Continuous removal of sludge from the primary tanks eliminates the need for sludge storage which can result in carry-over of additional solids in the primary effluent. Thickened sludge is pumped to three anaerobic digesters which are operated as single stage units with external heating and internal gas mixing. The units are sized to provide a minimum of 12 days detention time with a maximum loading of 0.21 pounds of volatile suspended solids per day per cubic foot. The digested sludge is withdrawn to two coil type vacuum filters, each having a surface area of 200 square feet. Lime and ferric chloride are added for sludge conditioning. Prior to up-grading to secondary treatment, the plant was using polymers for dewatering digested primary sludge. For years this filtered sludge had been disposed at no cost at a landfill site, a private horticultural operation (Bob Murphy, Inc., Vestal, New York), where the sludge was used as a soil conditioner. However, in 1974, Mr. Murphy informed the Binghamton-Johnson City Joint Sewage Board that the horticultural operation was ceasing and that the arrangement was no longer desirable. He has officially requested to the Joint Sewage Board reimbursement at the rate of $300 per month if continued handling were required. A temporary arrangement has been worked out for continuing disposal of sludge at this landfill site while other options are being investigated.

Costs for the existing sludge processing and disposal practices at the sewage treatment plant were evaluated during February 1974. The cost of chemicals for vacuum filtering

was \$31.29 per ton of dry solids using lime and ferric chloride for sludge conditioning. Adding labor cost to this, it was \$55.00 per ton of dry solids; adding hauling cost to Murphy's landfill site, it was \$60.39 per ton of dry solids. During February 1974, only 105 tons of sludge, based on dry solids, were removed. However, during March through May 1974, the average sludge removed was 165 tons of dry solids per month. At this rate, the cost of handling digested sludge will be approximately \$120,000 per year.

During January through June 1974, the cost of chemicals (lime and ferric chloride) for sludge conditioning varied from \$22.10 to \$42.00 per ton of dry solids; the average cost of chemicals was \$32.51 per ton of dry solids. The tests conducted during June and July with polymers for sludge conditioning showed promising results. During June and July, the computed costs per ton of dry solids were \$15.10 and \$15.26, respectively, using polymers for chemical conditioning.

At the sewage treatment plant, the following steps have been undertaken to reduce costs and to prepare alternative means of final disposal.

1. Experimentation with polymers for sludge conditioning to facilitate dewatering, thereby reducing the quantity of sludge to be hauled for disposal.

2. Cleaning the old drying beds to remove about 80 percent of the water remaining after filtering. This would decrease the quantity of sludge to be hauled for disposal, but would increase handling time and costs of the sludge.

3. Request to Southern Tier East Regional Planning Board for investigation of possible land application program for year-round disposal of liquid sludge on agricultural land. An application of liquid sludge would eliminate the costly method of vacuum filtering.

During January through May 1974, the monthly gas production from the digesters was approximately 3.9 million cubic feet or 0.13 million cubic feet per day. During the winter, 80 to 90 percent of the gas produced is used for heating the digesters. However, during the summer, only 30 to 40 percent of the gas produced is utilized in heating.

ENDICOTT SEWAGE TREATMENT PLANT

Secondary (trickling filters) sludge from two final clarifiers is sent to a wet well, where it is mixed with incoming raw sewage and then the mixture is allowed to settle in primary settling tanks. The sludge from the primary settling tanks is withdrawn continuously and distributed to two sludge thickeners. Concentrated sludge from both thickeners is withdrawn once per shift (3 shifts per day) and fed to a primary anaerobic digester; partially digested sludge from the primary digester is fed into a secondary anaerobic digester. Digested sludge from the secondary digester is withdrawn once per day and stored in a holding tank for dewatering. Ferric chloride and lime are added to the digested sludge for sludge conditioning. A vacuum filter is operated an average 1.5 days per week for dewatering. Dewatered sludge is hauled away in trucks to a landfill site about 1/4 mile from the Endicott sewage treatment plant.

The cost of chemicals for sludge conditioning is approximately \$12 per ton of dry solids. With labor costs, this increases to \$38.90 and the addition of a nominal fee for hauling to the landfill site increases the cost to \$39.20 per ton of dry solids. With approximately 2.6 dry tons removed per day, the average yearly cost would be \$40,000.

The Endicott landfill site, approximately 55 acres, is located on the north bank of the Susquehanna River. This landfill site is used for disposal of solid waste from the Village of Endicott and the Town of Union, as well as for the disposal of sludge from the Endicott sewage treatment plant. No information is available on the possible leaching of materials from the solid waste of sludge into the River.

VESTAL SEWAGE TREATMENT PLANT

The Vestal sewage treatment plant is a primary treatment system with sludge pumped to an anaerobic digester once per day. The sludge is then dewatered in drying beds. The dried sludge, about 0.3 tons of dry solids per day, is removed by farmers or other town individuals for use as a soil conditioner. There is a great demand for this digested sludge, so a problem of disposal has not occurred.

OWEGO SEWAGE TREATMENT PLANT No. 2

Owego sewage treatment plant No. 2 is an activated sludge (contact stabilization type) secondary treatment system. Primary sludge is pumped automatically, four times per day, to a sludge thickener. Excess secondary sludge is pumped to the same thickener. Thickened sludge is withdrawn once per day from the thickeners and fed to a primary anaerobic digester. This displaces partially digested sludge from the primary digester into a secondary anaerobic digester. Sludge is then withdrawn from the secondary digester, as necessary, and is removed by a tank truck for spraying on farm land. The plant operator makes arrangements with farmers for land application of liquid digested sludge. Usually there is a good demand for this digested sludge (about 0.4 tons of dry solids per day). If not, drying beds are available for use when needed.

During 1973, average digester gas production was approximately 68,000 cubic feet per month. Approximately 90 percent of the digester gas produced was utilized for heating the digester.

OWEGO SEWAGE TREATMENT PLANT No. 1

This is a trickling filter type secondary treatment system. Secondary sludge is pumped to a wet well for mixing with incoming raw sewage, which is allowed to settle in a primary settling tank. Primary sludge is withdrawn to a hopper and then fed, twice per day, to one anaerobic digester. Liquid digested sludge (about 0.2 tons dry solids per day) is hauled by tank trucks for land application. There is only one tank truck in use for both Owego Plants 1 and 2.

OWEGO VILLAGE--VILLAGE SEWAGE TREATMENT PLANT

This plant is currently a primary treatment system. Primary sludge is pumped once per day to a primary anaerobic digester and then to a secondary anaerobic digester. Liquid digested sludge (about 0.14 dry tons per day) is pumped to drying beds through perforated pipes. Supernatant from the bed is pumped to the head of the plant for mixing with incoming raw sewage. Dried sludge is

hauled to a landfill site near Candor. The gas produced in the digesters is not utilized for heating the digesters because of a malfunction of the heating systems. The digester gas is burned. The heating system will be redesigned during upgrading of the treatment plant to a secondary system.

OWEGO VILLAGE--VALLEY VIEW SEWAGE TREATMENT PLANT

This is an Imhoff tank serving only the small communities in the Village of Owego on the southern bank of the Susquehanna River. The Imhoff tank consists of a two-story tank in which sedimentation is accomplished in an upper compartment, and digestion in a lower compartment. Settling solids pass through slots into the unheated lower compartment and in adjoining vents. Gas produced in the digestion process in the lower compartment escapes through the vents and is an odor nuisance for the nearby area. The Valley View treatment plant will be phased out after the construction of a south side interceptor to transport the waste to the Village treatment plant. The construction of the south side interceptor is scheduled for 1976.

TECHNICAL EVALUATION OF SLUDGE MANAGEMENT PRACTICES

SLUDGE PROCESSING METHODS

Anaerobic Digestion

Anaerobic digestion has been used for a long time to reduce effectively the quantities of organic sludges and to transform them into stable, easily dewaterable residues. The reduction of organics is accomplished by the biological conversion of organic solids to methane and carbon dioxide. The gas produced has a high calorific content and is a valuable byproduct. The heat value of the gas produced from the digestion of sewage sludges ranges from 7,000 to 9,000 BTU per pound of organic matter destroyed, depending upon the nature and composition of the solids to be digested.

Methane gas produced during the gasification step is used for heating the digester to increase the rate of removal. The temperature of the digester is maintained around 85 to 90 degrees F. Polyelectrolytes or other chemical additives may be used to enhance sludge thickening. The anaerobic process content, ammonia content, mixing and loading; so for proper operation, these parameters must be carefully controlled. Since many toxic materials (heavy metals and sulfides for example) are concentrated in the sludge, digesters may be the first treatment units to be affected within a treatment system.

Digester gas contains about 65 to 70 percent methane by volume, 25 to 30 percent carbon dioxide, and small amounts of other gases. Its heating value is approximately 600 BTU per cubic foot. The digester gas can be utilized as a fuel for boilers and internal combustion engines which are, in turn, used for pumping sewage, operating blowers, and generating electricity. Hot water from boilers or from engine jackets may be used for sludge heating and for building heating.

Sludge Conditioning and Dewatering

Digested sludge is conditioned for the sole purpose of improving its dewatering characteristics. The two methods most commonly employed for conditioning of the sludge are the addition of chemicals and heat treatment. Ferric chloride, lime, alum, and polymers are the chemicals commonly used for conditioning. After conditioning, this digested sludge is dewatered either by vacuum filtration or by centrifugation. Mechanical dewatering by vacuum filters is applicable to all types of sanitary and industrial wastewater sludges. Centrifuges are becoming increasingly popular due to improvements in design. The moisture content of the centrifuged sludge is similar to that of sludge that has been dewatered by vacuum filtration, the solids content ranging from 15 to 25 percent in both cases. For small plants, it is a common practice to dewater and dry sludge on drying beds, with or without sludge conditioning. Digested sludge normally dries to 25 to 45 percent solids on drying beds.

The dewatered and partially dried sludge could be further dried by heat if necessary. The purpose of the heat drying is to remove additional moisture from the wet sludge, so that it can be incinerated efficiently or processed and fortified with nutrients into fertilizer.

FINAL DISPOSAL METHODS

Incineration

The incineration process is a natural extention of the drying process and converts the sludge into an inert ash, much reduced in volume (80-90% reduction), which can be disposed of easily. There are several types of incinerators on the market.

The multiple hearth furnace is one of the most successful and can be used for sludge drying as well as incineration. The furnace is a circular steel cylinder containing several hearths arranged in a vertical stack. Wet sludge (25-30% solids) is fed onto the top hearth and is slowly raked to the center. From the center, it drops into the second hearth where the racks move it to the periphery. Here it drops to the lower hearth and is again raked to the center. The hottest temperatures are on the middle hearths where the sludge burns and where auxiliary fuel is also burned, as necessary, to warm the furnace and to sustain the combustion. If the sludge is dewatered adequately (less than 70-75 percent moisture), and has the proper volatile content (70 percent or more), the process is usually self-sustaining, and no supplemental fuel is needed except for initial warm-up and heat control.

An important factor in multiple hearth furnace operation is the pollutants in the flue gas. It is important to reduce unburned hydrocarbons, oxides of nitrigen and particulates to an absolute minimum in the exhaust gases. The use of wet scrubbers, often preceded by pre-coolers, to remove particulates from exhaust gases is quite common. The use of wet scrubbers creates the problem of treating scrubber water. The scrubber water contains ash particles so it can be recirculated after particle collection by settling.

Land Application

Ultimate disposal of sludge commonly involves the land in many different forms: disposal of dried and dewatered sludge to a landfill site or for a soil conditioner; lagooning of sludge; and application to land in liquid form (no dewatering). The present discussion will focus on the land application of liquid sludge for use as a soil conditioner.

The difficulty and high cost of dewatering makes it advantageous to apply sludges to land in liquid form. The savings in avoiding dewatering must be traded against the added cost of transporting water associated with the sludge to the disposal site. Liquid sludge application is advantageous for two reasons: (1) water in the sludge serves as a source of irrigation water, and (2) nutrients and other dissolved solids are not recycled through the treatment plant. The digester supernatent, which is recycled to the beginning of the plant, contains ammonia and phosphates in high concentrations. The removal of these nutrients in liquid sludge would not only reduce loads to the treatment plant, but would also reduce the concentration of nutrients in the effluent discharged from the plant. Liquid sludge may be transported to the site by tank trucks or by pipelines. Railway tank cars of barges also have been used in some instances. Liquid sludge may be applied to soil by a tank truck equipped with sprinklers. Land application of sludge should be viewed as an alternate method of sludge disposal which recycles a natural resource, but not as a profitable agricultural venture. Sludge cannot compete with commonly available commercial fertilizers, because the major plant nutrients--nitrogen, phosphorus, and potassium--found in sludge are approximately one fifth the level found in chemical fertilizers. Sludge application also lacks the convenience and precise control available when using commercial fertilizers.

However, sludge has been widely used with good results on a variety of crops. Growth characteristics and yield have been comparable to those of crops receiving chemical fertilizers. Use of sludge is commonly avoided on crops to be eaten raw.

For continuous disposal of sludge on land, there is a need to provide holding tanks or alternate means of sludge disposal such as drying beds for expected interruptions due to weather or field conditions. Provision of sludge holding lagoons at the disposal site would serve this purpose and would enhance the pathogen elimination.

Suitable application rates are determined by the soil type, the nutrient and heavy metal content of the particular sludge and the nutrient uptake characteristics of the particular crop under consideration. Soils vary widely in permeability, pH, water table location, adsorption capacity, organic content, ion exchange, and chemical precipitation capability and thereby differ in their capacity to assimilate sludges. A maximum slope of 5 percent is recommended. Extra care should be taken when dealing with acid soils, because of

the possible release of heavy metals. As a general rule, a pH above 6.5 is recommended to control heavy metal solubility. The loss of nitrogen and phosphorus from the disposal site are of major concern because of the association of these two nutrients with the eutrophication of surface water and because of possible contamination of groundwater with nitrates.

Careful consideration should be given to site selection for land application: isolation from residences, as well as from surface streams. Some soils may restrict the solids application rate to only a few tons per acre per year. However, the reported maximum rates vary by as much as a factor of 10, depending on the soil characteristics, heavy metal concentrations, nutrient content, and crop planted.

Anaerobic Digestion of Sludge with Municipal Solid Waste

Various studies have demonstrated that organic refuse, particularly garbage, is amenable to anaerobic decomposition as long as proper conditions of moisture, temperature, pH, absence of oxygen and toxic materials, and adequate nutrients are maintained. Organic refuse is generally deficient in nutrients, so the addition of sewage sludge in proper proportions may provide the required nutrients as well as readily degradable organic material and active micro-organisms.

Municipal solids wastes are usually divided into residential wastes, commercial wastes, demolition and construction wastes, and special wastes (street refuse, treatment plant residue, dead animals, automobiles and tires, and hospital refuse). Except for treatment plant residues, some fraction of each of the categories listed is usually collected and handled at disposal sites.

A sub-classification of municipal solids wastes, usually termed "combined refuse" is relevant to this discussion on anaerobic digestion and is composed of selected household wastes (household garbage and rubbish, lawn clippings and prunings); commercial wastes (refuse from stores, markets, offices, schools, airports, etc.); and street refuse (sweepings, leaves, tree trimmings). Combined refuse is usually estimated to be generated at the rate of 4 to 6 pounds per capita per day, depending on the size of the community. Of this amount, it can be expected that 50 percent will be paper, 10 percent will be garbage, and 10 percent will be leaves and trimmings. Some 30 percent can be expected to be metals, glass, and other materials unsuitable for biological degradation, but which cannot be easily separated from the organic refuse during collection.

The same basic compounds (carbohydrates, proteins, and fats) characteristic of sewage sludge will comprise most of the organic material.

Anaerobic digestion of organic refuse would not only reduce the volume of refuse requiring disposal, but would provide excess methane gas as a by-product. Assuming that about 70 percent of combined refuse is biodegradable and that 70 percent of the biodegradable portion is gasified in an anaerobic process, the resulting overall reduction in weight of dry solids would amount to about 50 percent. This would greatly reduce hauling costs to the landfill sites and would extend the life of the sites, if the solids from the digester effluent could be effectively dewatered.

The addition of non-digested sewage sludge, at the level of 3 to 6 percent by weight of total organic solids to be digested, improved the digestion process for organic refuse even when adequate supplemental nutrients (nitrogen and phosphorus) were added. Apparently, trace nutrients in the sludge as well as a continued innoculation with new micro-organisms were responsible for the improved digestion. (Pfeffer, 1973).

The recent study "Consolidation of Refuse Collection and Disposal Services in Broome County," (March 1974), conducted by the State University of New York at Binghamton for Southern Tier East Regional Planning Board, reports that 202,000 tons per year of municipal solid wastes are collected at seven landfill sites in the County (Table VI-2).

Two of these seven sites are county-operated, and the other five sites are municipally operated. Approximately 59 percent of the total reported waste is collected at the Nanticoke site, and approximately 21 percent is collected at the Endicott site. The total waste collected at these two sites amounts to 882 tons per day. Assuming 60 percent of this waste is biodegradable and postulating the addition of non-digested sludge at the level of 3 percent of total organic solids to an anaerobic digestion process, the required quantity of sludge would be approximately 16 tons per day of dry solids. By the year 2000, the refuse collected per capita would be expected to increase to 7 lbs/cap/day, while the population in Broome County is expected to increase by 9 percent. Thus, the quantity of sewage sludge required to augment anaerobic digestion of solid wastes would increase to approximately 24 tons per day. This compares reasonably well with the quantity of raw sludge expected to be generated by the Binghamton-Johnson City, Endicott, Vestal, Nanticoke Valley, Chenango Valley, and Five Mile Point Management Areas--20 tons per day dry solids by the year 2000.

TABLE VI-2

AMOUNT OF SOLID WASTE DISPOSED TO LANDFILL SITES IN BROOME COUNTY, 1973

Location	Reported Annual Tonnage	Tonnage per day**	Percent
County-Run Sites			
Colesville	19,000	52.1	9
Nanticoke	119,915	328.5	59
Other Public Sites			
Chenango	10,298 *	28.2	5
Conklin	4,927 *	13.5	2
Endicott	42,399 *	116.2	21
Fenton	6,000	16.4	3
Lisle	1,750*	4.8	1
TOTAL	202,000	553.4	100

*Estimated Annual Tonnage
**Based on 365-day operation.

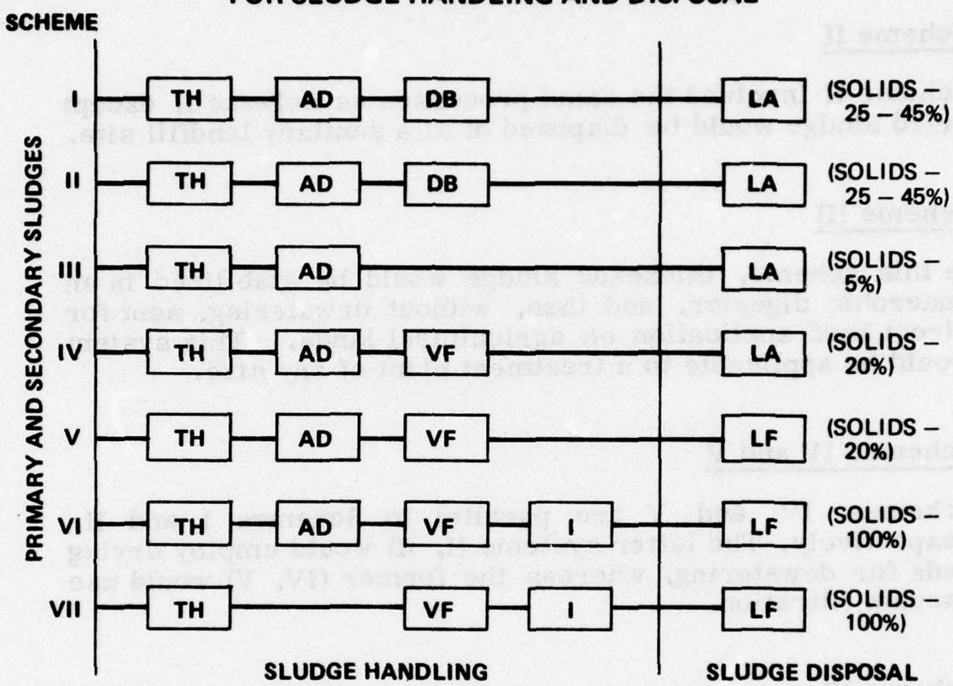
PRELIMINARY SCHEMES FOR SLUDGE MANAGEMENT

DESCRIPTION OF SYSTEMS

Several sludge handling schemes were formulated for the purpose of cost comparison and are shown schematically in Figure VI-1. The cost comparisons developed were for a generalized situation and are presented for ranges of plant sizes.

Most of these sizes exceed the 1 to 2 mgd upper limit normally associated with aerobic digestion. Moreover, all treatment plants presently operating in the subject area utilize anaerobic digestion. The aerobic process was, therefore, not included in any of the schemes.

PRELIMINARY SCHEMES FOR SLUDGE HANDLING AND DISPOSAL



TH – THICKENER
AD – ANAEROBIC DIGESTER
DB – DRYING BEDS
VF – VACUUM FILTER
I – INCINERATOR
LA – LAND APPLICATION
LF – LAND FILLING

FIGURE VI-1

Scheme I

In this scheme, primary and secondary sludges would be transferred first to a thickener and then to a digester. Digested sludge would be pumped to sludge drying beds and spread with the help of perforated pipe. Dried sludge (solid concentration 25 to 45 percent) would then be trucked away for land application. This system is mainly used for small to medium size capacity (2 to 5 mgd) plants.

Scheme II

Scheme II involved the same processes as Scheme I, except dried sludge would be disposed of at a sanitary landfill site.

Scheme III

In this scheme, thickened sludge would be stabilized in an anaerobic digester, and then, without dewatering, sent for direct land application on agricultural lands. This system would be applicable to a treatment plant of any size.

Schemes IV and V

Schemes IV and V are parallel to Schemes I and II, respectively. The latter systems (I, II) would employ drying beds for dewatering, whereas the former (IV, V) would use vacuum filtration.

Scheme VI

In this scheme, digested, dewatered sludge would be incinerated with the ash disposed on a landfill site. Incineration is used in medium to large size plants in urban areas. Due to air quality regulations, the incinerators require special air pollution control devices, increasing the cost of operation.

Scheme VII

In Scheme VII, instead of digested sludge, non-digested, dewatered sludge would be incinerated directly and the ash disposed of on a landfill site.

Scheme VIII

In this scheme (not shown on Figure VI-1), non-digested sludge would be added to an organic portion of municipal refuse for stabilization in an anaerobic digester. The digested sludge would be dewatered with the help of chemicals and then trucked to a landfill for ultimate disposal. The digester gas generated could be utilized for its heat value.

COST COMPARISON

Costs developed for Schemes I through VII were made on the generalized basis of treatment plant size and sludge production. The comparisons were, therefore, independent of specific treatment plants or specific wastewater management alternatives. No comparable cost information was available for Scheme VIII, incorporating municipal refuse with sludge, due to the dependence of this cost on specific site analysis.

Costs for Schemes I through VII, based on characteristics of average treatment processes, are presented in Table VI-3. Costs for land are not included. Systems employing land application were assumed to use existing agricultural land rather than purchasing it.

In Schemes I and II, the cost of land required for sludge drying beds was not included, so actual costs would be slightly higher than presented. With an increase in plant capacity, the size of the sludge drying beds would likewise increase. The unit price of land would also increase, due to a change in the site location of the plant from a rural to an urban area. So the cost advantage of sludge drying beds for small plants diminishes with increasing plant size. Comparison of the costs for Schemes I and IV indicates that for small size plants, drying beds are less costly than vacuum filters. However, as the size of the plant increases to about 5 mgd, the costs for the two systems become comparable. In larger treatment plants, sludge drying beds become impractical due to the volume of sludge and the amount of land needed for drying.

For small size plants, the cost difference between land application (Schemes I and IV) and land filling (Schemes II and V) of digested, dewatered sludge would not be significant.

TABLE VI-3

COSTS FOR TYPICAL SLUDGE MANAGEMENT SCHEMES[1]

Sludge (Tons Dry Solids/Day)	Equivalent Wastewater Flow (mgd)	Scheme I	II	III	IV	V	VI	VII
		Capital Costs ($1,000)						
0.5	0.8	120	120	87	277	277	450	377
1.0	1.5	197	197	138	423	423	682	572
5.0	7.7	1,138	1,138	523	941	941	1,602	1,214
10.0	15.0			1,014	1,860	1,860	2,850	2,247
20.0	31.0			1,886	2,913	2,913	4,396	2,997
		Operating Costs ($1,000/year)						
0.5	0.8	18	18	9	26	26	37	32
1.0	1.5	30	31	15	40	41	57	47
5.0	7.7	118	122	61	125	129	183	148
10.0	15.0			116	223	232	326	263
20.0	31.0			223	403	421	591	471

[1]See Figure VI-1 for Strategy description.
Based on generalized performance criteria and treatment process costs and not on study area characteristics. Cost of land not included.
Round trip distance to disposal site assumed to be 30 miles.

Comparing Scheme I with II and Scheme IV with V, the cost of land application and landfill are comparable. However, when the cost of land is included, landfill becomes more expensive than land application. For these cost comparisons, the distance for all schemes was selected as 30 miles. However, for small size plants, the distance for any method of sludge disposal will usually be quite short.

The costs for incinerating either the digested, dewatered sludge (Scheme VI) or nondigested, dewatered sludge (Scheme VII) are comparatively higher than costs for other systems. This is due to the high capital investment for the incinerator. With an increase in plant size, the relative costs for incineration decrease considerably.

The direct land application of liquid digested sludge (Scheme III) is very cost effective in comparison with all other systems. The cost effectiveness of Scheme VIII, digestion of municipal refuse and sludge, could be realized in the value of the energy generated and the decrease in sludge and refuse disposal costs.

An important aspect in comparing these general schemes is that the costs for the same system vary considerably depending on the specific site and circumstances. For example, a survey conducted by EPA of the costs for land application of liquid digested sludge in the northwestern counties of Ohio, found considerable cost variations: hauling by contract *would cost on the average*, $31.92 per ton of dry solids, and hauling by city-owned trucks would cost $7.73 per ton of dry solids (Manson and Merritt, 1973).

FORMULATION OF SLUDGE MANAGEMENT ALTERNATIVES

DEVELOPMENT OF SPECIFIC ALTERNATIVES

The eight schemes previously outlined for the purpose of generalized cost comparisons were examined to select the most promising for detailed analysis.

Schemes I and II utilized sludge drying beds, rather than vacuum filtration, for dewatering in small treatment plants. These sand beds are used infrequently at both Town of Owego STP's No. 1 and 2, but are used exclusively at the

Owego Village STP. However, since both the Binghamton-Johnson City and the Endicott STP's presently have vacuum filters and future plans call for one to be installed at Owego Village, sand beds were eliminated from the analysis.

Both Schemes III and IV employ land application of sludge as a soil conditioner. However, with the latter, sludge would be dewatered prior to application. Because of the difficulty and high cost of dewatering processes, Scheme IV was not further considered.

Schemes VI and VII both used incineration for final disposal. The increased cost of including a digester for Scheme VI without compensating benefits, eliminated this system from further study.

Scheme VIII, digestion of sludge with solid waste, was also eliminated because of the lack of available data on cost and dependability, and because of the comparative simplicity with which the other alternatives can be implemented.

The sludge management alternatives selected for specific analysis were basically similar to Schemes III, V, and VII. These alternatives, depicted in Figure VI-2, include:

Alternative A -- Incineration

Alternative B -- Application of Liquid Sludge to Agricultural Land

Alternative C -- Landfill of Dewatered Sludge

The analysis of the three alternatives considered the actual existing conditions, i.e., sludge quantities generated at each plant, the processing capabilities of each plant and current methods of sludge disposal. Existing disposal sites were also used in the analysis.

DESIGN PARAMETERS

Sludge Handling Processes

Because certain alternatives required the addition of particular unit processes and/or additions to the existing facilities, Table VI-4 illustrates the design parameters utilized for these processes. Alum sludge is difficult to dewater, hence the use of a lower loading rate on the vacuum filter for both Bio AWT and P/C AWT.

FLOW SHEETS FOR SLUDGE MANAGEMENT ALTERNATIVES

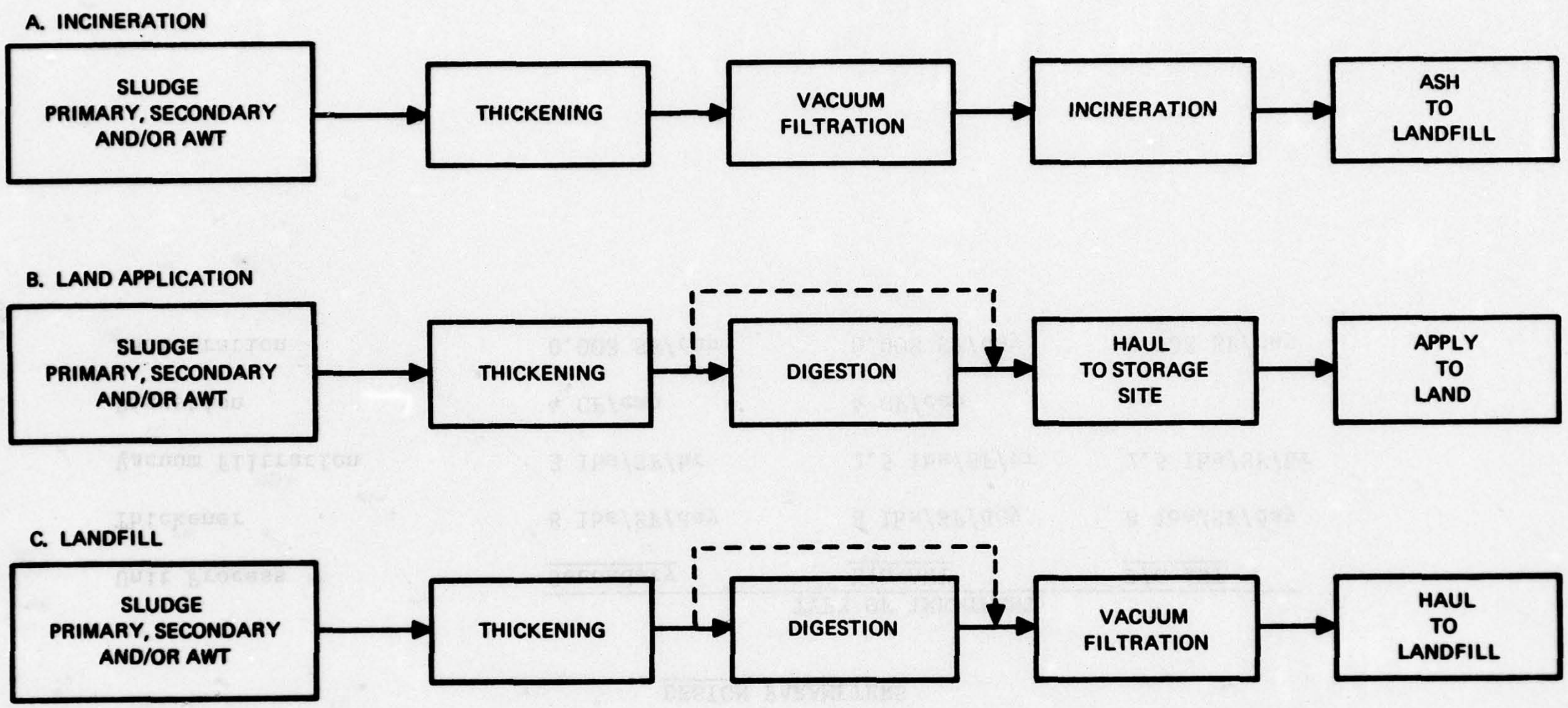


FIGURE VI-2

TABLE VI-4

SLUDGE HANDLING PROCESSES

DESIGN PARAMETERS

Unit Process	TYPE OF TREATMENT Secondary	Bio-AWT	P/C AWT
Thickener	8 lbs/SF/day	8 lbs/SF/day	8 lbs/SF/day
Vacuum Filtration	3 lbs/SF/hr	2.5 lbs/SF/hr	2.5 lbs/SF/hr
Digestion	4 CF/cap	4 CF/cap	---
Incineration	0.008 SF/cap	0.008 SF/day	0.008 SF/cap

Hauling Costs

For the purpose of computing the number of trips per year for hauling sludge, two different trucks are assumed available: that is, for hauling liquid sludge to the land application site, a 6,000 gallon tanker truck is used; for hauling the dried sludge or ash to the landfill area, a 10-ton dump truck is utilized. In conjunction with this, an assumption is made regarding the distance to and from the landfill site. The Broome County landfill site is located on Knapp Road in the Town of Nanticoke, northwest of the Broome County Airport. This is approximately 25 miles and 18 miles round trip from the Binghamton-Johnson City and Chenango Valley STP's, respectively.

The Endicott STP has its own landfill site which is about a 0.5 mile round trip distance. For the remaining plants, East Owego and West Owego, due to the rural nature of the area, it was assumed that a landfill site could be found within a 5-mile radius of each plant. Therefore, the round trip distance is considered to be 10 miles. The quoted distances for each treatment plant were used in computing the number of miles per year and subsequently the number of gallons of truck fuel per year required (@ 6 mi/gal). The hauling charge, additional to fuel costs, was assumed to be a flat rate of $1/dry ton solids/mile.

Required Landfill Acreage

In order to calculate the amount of acreage required for landfill disposal of the sludge, the following assumptions are made. Typically, a sanitary landfill consists of trenches, whereby sludge is deposited daily and covered by a 6-inch layer of sand or soil. After three feet of sludge has accumulated, a two foot layer of soil is placed on top. This is followed by another three foot layer of sludge and an addtional two feet of soil. Therefore, in every acre of land, a landfill can accommodate 261,000 cubic feet (6 feet x 43,566 square feet) of sludge.

Assuming the dewatered sludge weighs more than water at 62.4 lbs/cubic foot and less than sand at 105 lbs/cubic foot, a value of 75 lbs/cubic foot is taken as being representative of sludge. Utilizing this figure yields the amount of acreage required for disposal.

Incineration produces an inert ash that contains no water. For this reason, it is assumed to weigh approximately 1.75 times as much as dewatered sludge or 130 lbs/cubic foot. This value is then used to calculate the required acreage for ash disposal based on the 261,000 cubic foot/acre figure stated previously.

Land Application Design Loading Rate

Several factors effect the actual loading rate and consequent design of a land application system, such as: heavy metal content of the sludge, soil acidity, type of crops grown, and nutrient content of the sludge.

The heavy metal content of sludge is a major controlling factor in the design of a land application system. Essentially, this is because it is unknown what the long term effects of heavy metals are on the soil, crops, and food chain. However, it is known what factors do affect the toxicity of the metals to plants. These are: background concentration of metals, the metals present in the sludge, pH of the soil (more toxic at pH greater than 6.5), organic matter content (organic matter chelates the metals and makes them less available), phosphate content (binds up the lead) cation exchange capacity (CEC binds up cations), and finally, the type of crop grown (grains and corn less sensitive than vegetative crops).

Since the organic matter and phosphate content of sludge is relatively high, this should help counteract the toxic effect. In the Broome-Tioga County area, the soils are largely clay in nature, thus possessing a rather high cation exchange capacity. This will tie up a good percentage of the heavy metals. However, the soils in the Study Area are also rather acidic, which infers a greater toxicity effect. Since lime is normally added as a fertilizing agent to farmland, this effect would be mitigated. The crops grown in the Broome-Tioga County region are typically grains and corn which are less sensitive than other crops to heavy metals.

Because the actual heavy metal content of the sludge is unknown in the Broome-Tioga County area, it was assumed to be typical of most urban centers. Table VI-5 presents typical values for heavy metal content in sludge. These concentrations are relatively high and must be considered in the final selection of an application rate.

TABLE VI-5

HEAVY METAL CONTENT IN SLUDGE

All Values in PPM

Metal	Chaney Range	Blakeslee Range	Mansest, et al. Ave	Klein, et al. Ave	EPA Range	Tallander Range	Blakeslee Ave	Sludge Without Industrial Pollution after Chaney
Hg	<1-10	1.9-56	--	--	--	4-8	8.7	<15
Cr		22-9600	880	290	--	50-200	1290	--
Cu	250-17,000	260-10.400	26	460	720-6020	500 1500	1260	<800
Ni	25-8000	20-1200	190	70.2	<200-600	25-100	293	<100
Fe	--	1298-44,000	--	--	--	--	17,300	--
Zn	500-50,000	1120 16,400	64	658	1700-9000	1000-3000	3390	<200
Cd	5-2000	2-1100	0.6	24	<40-830	8-15	116	<10
Pb	100-100,000	240-12,400	--	--	--	100-300	1340	<1000
As	--	4-18	--	--	--	--	9.0	--
B	15-1000	--	--	--	--	--	--	<1000
Mn	--	--	--	--	--	200-500	--	--
Co	--	--	--	--	--	8-20	--	--

The nutrient content of sludge depends to a large extent on the degree of treatment. The following N:P:K ratios were utilized in predicting the nutrient content of the various types of sludge on a percent dry solids basis: raw primary plus secondary--5. 6:2. 9:0. 4; digested or dewatered primary plus secondary--2:1:0. 2; dewatered, digested primary plus secondary--1. 5:0. 75:0. 1.

All of the aforementioned factors must be accounted for in the selection of an application rate. For this report, a rate of 5 tons dry solids/acre/year was used based on these factors. (See detailed discusssion of application rates in Chapter V of the Speciality Appendix.)

PROJECTED SLUDGE QUANTITIES

In general, sludge quantities for the year 2020 were determined on a pound/capita/day basis. This figure was obtained from either existing conditions, if known, or based on typical design values.

The amount of sludge prodced is also a function of the level of wastewater treatment and, therefore, the analysis was sub-divided accordingly. Projected sludge quantities for the year 2020 are presented in Table VI-6.

Sludge from Secondary Treatment Processes

Binghamton-Johnson City STP data was utilized to estimate sludge production. The total primary plus secondary sludge produced at this plant is 42, 500 lbs/day. The plant's digester reduces the total solids content by approximately 30 percent which yields digested sludge of 29, 300 lbs/day. The digested and undigested sludge quantities yield a lb/cap/day production of 0. 286 and 0. 415, respectively. These numbers, applied to the population projections for the year 2020, produce the sludge quantities shown in Table VI-6.

The Endicott STP currently generates approximately 3, 000 lbs/day of digested sludge or 0. 067 lb/cap/day. This figure is relatively low because the influent suspended solids are also low (0. 13 lb/cap/day). Based on a typical value for treatment plants of 40 percent reduction in the digestion process, the amount of undigested primary and secondary sludge produced is 5, 000 lbs/day or 0. 089 lb/cap/day. These figures were scaled up to the 2020 population numbers and are presented in Table VI-6.

TABLE VI-6

SLUDGE QUANTITIES FOR YEAR 2020
(lbs/day)

STP	Secondary*	Biological AWT	Physical/ Chemical AWT
	Type of Wastewater Treatment		
Binghamton-Johnson City			
Digested	35,400	50,100	**
Undigested	51,300	68,600	75,600
Endicott			
Digested	5,000	11,800	**
Undigested	8,400	16,000	14,900
East Owego			
Digested	2,800	4,900	**
Undigested	4,800	7,100	5,800
West Owego & Owego Village			
Digested	2,000	3,400	**
Undigested	3,400	5,000	4,100
Chenango Valley			
Digested	3,200	5,400	**
Undigested	5,300	7,900	6,400
TOTALS			
Digested	48,400	75,600	--
Undigested	73,200	104,600	106,800

*Same for secondary plus nitrification.

**Digestion process is not used for physical/chemical sludges.

Since no sludge quantity data exists for the remaining service areas (East Owego, West Owego (with Owego Village) and Chenango Valley), the following equation from the literature was utilized for sludge production:

Sludge Production = 0.85 (influent suspended solids)

+0.5 (0.65)(influent BOD)

where: Inf. SS = 0.2 lb/cap/day

Inf. BOD = 0.17 lb/cap/day

0.65 = BOD applied to secondary treatment process

0.5 = growth associated with BOD removal

0.85 = total SS removal

This equation resulted in a lb/cap/day input of 0.225 for raw, undigested sludge. Applying a typical value of 40 percent reduction in the digestion process yielded 0.135 lb/cap/day of digested sludge. These numbers give the sludge quantities for the year 2020 illustrated in Table VI-6. These values also apply to secondary plus nitrification treatment plants as little sludge is wasted in the nitrification process.

Biological AWT Sludge

In addition to the primary and secondary sludge produced, biological AWT would contribute slightly more biological sludge (from denitrification), a considerable amount of chemical sludge (from phosphorus removal) and some sludge from the filtration unit.

The sludge formed from the nitrification process is considered negligible since wasting is performed infrequently. Denitrification, however, generates sludge at the rate of 0.7 pounds per pound of nitrate (NO3-N) removed. Applying the NO3-N values utilized in the design of the denitrification system in conjunction with the above-mentioned relationship resulted in the lbs/day of sludge produced from the denitrification process.

Filtration generates sludge in its backwashing operation and these solids are recycled back to the primary clarifier for ultimate removal. Since it is unknown what percentage of the influent suspended solids would be removed in the biological processes, it has been assumed that the filtration

solids are equivalent to 10 percent of the influent SS load. This yields a lb/cap/day production of filtration solids of 0.0537 at Binghamton-Johnson City, 0.013 at Endicott, and 0.02 at the remaining treatment plants.

The amount of chemical sludge was based on a strict stoichiometric relationship and was estimated to be 0.0715 lbs/cap/day.

The three aforementioned sludge production numbers (denitrification, chemical, and filtration) were used in computing the additional sludge produced by Bio-AWT. These numbers, when added to those previously calculated for secondary treatment, resulted in the total sludge quantity for each biological AWT plant in the year 2020 as shown in Table VI-6. It should be noted that the digested sludge figures were calculated on the basis of the previously mentioned reductions in total solids. However, the chemical solids were eliminated in the computations for digested quantities as the anaerobic digestion process cannot be used for chemical sludges.

<u>Physical/Chemical (P/C) AWT Sludge</u>

In the P/C AWT process, all of the organic sludge produced would be generated in the primary clarifier. Because of this, the sludge formed by P/C AWT was assumed to be equivalent to the per capita input of suspended solids. Of this figure, 15 percent was assumed to be due to the filtration process while the remaining 85 percent resulted from the addition of alum in the clarifier. Chemical sludge would be created in the same quantities as were established for Biological AWT.

Based on the above, sludge quantities were projected in Table VI-6. As previously mentioned, the digestion process is not used for physical/chemical sludges.

ANALYSIS OF ALTERNATIVES

This section analyzes each alternative considered (incineration, landfill, and land application) on the basis of cost-effectiveness, impact assessment and evaluation. Recommendations regarding future sludge handling and disposal practices in the Broome-Tioga County area are also given.

Alternative A: Incineration

Alternative A, Incineration, involved (1) thickening of the sludge by a gravity thickener, (2) dewatering by vacuum filtration, (3) incineration in a multiple hearth incinerator, and (4) hauling the ash by truck to a landfill for disposal. Because the incineration process and related costs depend to a high degree on the percent moisture content of the sludge, it was necessary to include both thickening and dewatering processes. The combination of these two processes could reduce the moisture content of the sludge from approximately 99 percent to about 75 percent.

The inert residue or ash produced in the incineration process must be disposed of at a landfill. Because incineration involves evaporation of the water content in the sludge, the residue contains no water and so the volume to be disposed of would be relatively minor (10-20%) in comparison with the non-incinerated sludge. Approximately 14-30 acres of land would be needed over the 50-year life of the project depending on the level of wastewater treatment. This system pertains to all wastewater treatment alternatives considered: that is, secondary, secondary plus nitrification, biological AWT, and P/C AWT.

Alternative B: Land Application

Land application of sludge required (1) gravity thickener, (2) anaerobic digestion, (3) hauling of the liquid sludge by tanker truck to a storage site, and (4) application of the sludge on agricultural land.

Thickening reduces the moisture content of the sludge and consequently the volume to be handled. Digestion reduces the volatile content of the sludge and so the volume to be disposed of. Digestion also destroys pathogenic organisms associated with the sludge. Digestion was not used in the flow diagram for P/C AWT as the volatile content would be too low and the organaic sludge could not be separated from the chemical sludge.

Regarding biological AWT, the sludges from the major unit processes could be easily separated. The one exception--the denitrification sludge--would be insignificant in comparison with the primary and secondary sludge. Therefore, the flow diagram for biological AWT included digestion of the primary and secondary sludges and land application of these sludges together with the denitrification and chemical sludges.

The sludge would be hauled without dewatering since it is more easily applied to the land and is more readily plowed into the soil by the farmer. Depending upon the level of wastewater treatment, approximately 1,770 to 2,825 acres of land would receive sludge application. As existing agricultural land would be utilized, and because there appears to be a good demand for sludge as a soil conditioner, purchase of land would not be required.

Tank trucks would be used to haul the sludge because of its consistency (approximately 95 percent water). Storage of the sludge would also be required because of the seasonal nature of the application, i.e., the growing season (April-September). Aproximately 7 to 16 acres of land would be necessary for storage lagoons.

Alternative C: Landfill

Alternative C included (1) gravity thickener, (2) anaerobic digestion, (3) dewatering by vacuum filtration, and (4) hauling of the sludge by dump truck to a landfill site.

Again the thickening and digestion processes would be used to reduce the amount of sludge to be handled. The digestion process would not be used for physical/chemical or denitrified sludges. Vacuum filtration, as stated previously, would reduce the moisture content to 75 percent, thus reducing the volume to be handled. Dump trucks would be used to haul the sludge to the landfill site. Over the 50-year project life, 226 to 425 acres of land would be purchased and committed to landfill, depending on the degree of wastewater treatment.

COST ANALYSIS

Tables VI-7 through VI-15 present both the capital and operating costs for the sludge management systems for the three types of wastewater treatment in each service area. The capital costs reflect those unit processes which either need extentions or would be required as a complete addition to the treatment plant. These costs were derived from cost curves contained in either the Black and Veatch or the EPA sludge handling manual and were scaled up to 1974 dollars. Engineering and contingencies were also included. Likewise, O&M costs were obtained from the above mentioned sources and also scaled up to 1974 dollars. Land costs associated with landfill disposal were computed at a cost of $500/acre which is representative of the area. Land costs were not included for land application since it was assumed existing farmland would be used. Also, since this alternative is reversible (because of the small capital commitment), it would not be necessary to purchase the land from the farmers. Approximately 7 to 16 acres, however, would be purchased for storage lagoons.

In order to determine the cost-effectiveness of each sludge handling alternative, the total average annual cost is presented. Costs were based on a 50-year time scale and a 6 1/8 percent interest rate. Replacement costs were based on a 25-year useful life period of each unit. Existing units were accounted for in computing the replacement cost.

As indicated in the tables, Alternative B, Land Application, would be the least costly system for each level of treatment and for each service area. Alternative C, Landfill, would be the next least expensive system, with costs significantly lower than Alternative A, Incineration. This is because of the large investment required for both the vacuum filter and incineration units.

TABLE VI-7

INCINERATION COSTS – ALTERNATIVE A[1]

SECONDARY TREATMENT

Incineration	**B-JC**	**Endicott**	**E. Owego**	**W. Owego[2]**	**Chenango Valley**	**Total (Million)**
Capital ($ Million)	2,300,000	760,000	610,000	610,000	610,000	4.9
Annual Cap. & Replacement @ .081($/Yr)	192,600	63,700	50,700	50,700	50,700	0.41
O&M ($/Yr)	109,000	38,500	29,500	26,500	31,400	0.23
Total Annual ($/Yr)	301,600	102,200	80,200	77,200	82,100	0.64
Vacuum Filtration						
Capital ($)	1,200,000	–	360,600	230,000	380,000	13.0
Annual Cap. & Replacement @ .019 ($/Yr)	114,800	12,900(R)	35,200	22,800	37,300	0.22
O&M ($/Yr)	223,000	52,400	38,000	30,300	42,900	0.39
Total Annual ($/Yr)	337,800	65,300	73,200	53,100	80,200	0.61
Total						
Total Annual ($/Yr)	639,400	167,500	153,400	130,300	162,300	$\boxed{1.25}$
Present Worth ($Million)	9.9	2.6	2.4	2.0	2.5	$\boxed{19.4}$

[1] 6-1/8% @ 50 years

[2] Includes Owego Village

TABLE VI-8

LAND APPLICATION COSTS – ALTERNATIVE B[1]

SECONDARY TREATMENT

Digester	B-JC	Endicott	E. Owego	W. Owego[2]	Chenango Valley	Total ($ Million)
Capital ($)	–	–	–	–	460,000	0.46
Annual Cap. & Replacement ($/Yr)	17,600	9,800	8,300	6,700	38,300	0.08
O&M ($/Yr)	25,400	16,000	13,000	12,500	15,000	0.08
Total ($/Yr)	43,000	25,800	21,300	19,200	53,300	0.16
Haul @ $32/ton (dry) ($/Yr)	207,000	29,000	17,000	12,000	19,000	0.28
Total Annual ($/Yr)	250,000	54,800	38,300	31,200	72,300	$\boxed{0.45}$
Present Worth ($ Million)	3.87	0.85	0.59	0.48	1.12	$\boxed{6.9}$

[1] 6-1/8 @ 50 years

[2] Includes Owego Village

TABLE VI-9

LANDFILL COSTS – ALTERNATIVE C[1]

SECONDARY TREATMENT

	B-JC	Endicott	E. Owego	W. Owego[2]	Chenango Valley	Total ($ Million)
Digester						
Capital ($)	–	–	–	–	460,000	0.46
Ann. Cap. & Replacement ($/Yr)	17,600	9,800	8,300	6,700	38,300	0.08
O&M ($/Yr)	25,400	16,000	13,000	12,500	15,000	0.08
Total ($/Yr)	43,000	25,800	21,300	19,200	53,300	0.16
Vacuum Filtration						
Capital ($)	–	–	260,000	170,000	280,000	0.71
Ann. Cap. & Replacement ($/Yr)	9,300	9,300	21,700	14,500	23,800	0.08
O&M ($/Yr)	160,000	39,300	27,600	22,800	30,200	0.28
Total Ann. ($/Yr)	169,300	48,600	49,300	37,300	54,000	0.36
Haul @ 81/ Wt - ton						
Annual ($/Yr)	26,000	3,600	2,100	1,500	2,300	0.04
Landfill						
Capital ($)	340,000	100,000	60,000	55,000	75,000	0.63
Ann. Cap & Replacement ($/Yr)	28,500	8,400	5,100	4,700	6,300	0.05
O&M ($/Yr)	95,000	29,000	24,000	19,000	23,500	0.19
Total Ann. ($/Yr)	123,500	37,400	29,100	23,700	29,800	0.24
Grand Total ($/Yr)	361,800	115,400	101,800	81,700	139,400	0.80
Present Worth ($ Million)	5.6	1.79	1.58	1.27	2.16	12.4

[1] 6-1/8% @ 50 years

[2] Includes Owego Village

TABLE VI-10

INCINERATION COSTS – ALTERNATIVE A[1]

BIO-AWT

	B-JC	Endicott	E. Owego	W. Owego[2]	Chenango Valley	Total ($ Million)
Incineration						
Capital ($)	2,500,000	1,230,000	720,000	610,000	740,000	5.8
Ann. Cap. & Replacement ($/Yr)	207,100	103,500	60,500	50,700	62,100	0.48
O&M ($/Yr)	141,000	56,500	36,400	27,400	38,900	0.30
Total Ann. ($/Yr)	348,100	160,000	96,900	78,100	101,000	0.78
Vacuum Filtration						
Capital ($)	2,280,000	–	446,000	360,000	490,000	3.58
Ann. Cap & Replacement inc. exist. ($/Yr)	204,000	12,900	37,300	35,200	41,400	0.33
O&M ($/Yr)	270,000	93,000	51,600	38,000	52,400	0.50
Total Ann. ($/Yr)	474,000	105,900	88,900	73,200	93,800	0.84
Total Annual ($/Yr)	822,100	265,900	185,800	151,300	194.800	$\boxed{1.62}$
Present Worth ($ Million)	12.73	4.12	2.88	2.34	3.02	$\boxed{25.1}$

[1] 6-1/8% @ 50 years

[2] Includes Owego Village

TABLE VI-11

LAND APPLICATION COSTS – ALTERNATIVE B[1]

BIO-AWT

	B-JC	Endicott	E. Owego	W. Owego[2]	Chenango Valley	Total ($ Million)
Digester						
Capital ($)	–	–	–	–	460,000	0.46
Annual & Replacement ($/Yr)	17,600	9,800	8,300	6,700	38,300	0.08
O&M ($/Yr)	25,400	16,000	13,000	12,500	15,000	0.08
Total ($/yr)	43,000	25,800	21,300	19,200	53,300	0.16
Haul @ $32/ton dry ($/Yr)	293,000	68,400	28,500	20,000	31,700	0.44
Total Annual ($/Yr)	336,000	94,200	49,800	39,200	85,000	$\boxed{0.60}$
Present Worth ($ Million)	5.20	1.46	0.77	0.61	1.32	$\boxed{9.3}$

[1] 6-1/8% @ 50 years

[2] Includes Owego Village

TABLE VI-12

LANDFILL COSTS – ALTERNATIVE C[1]

BIO-AWT

	B-JC	Endicott	E. Owego	W. Owego[2]	Chenango Valley	Total ($ Million)
Digester						
Capital ($)	–	–	–	–	460,000	0.46
Ann. Cap. & Replacement ($/Yr)	17,600	9,800	8,300	6,700	38,300	0.08
O&M ($/Yr)	25,400	16,000	13,000	12,500	15,000	0.08
Total ($/Yr)	43,000	25,800	21,300	19,200	53,300	0.16
Vacuum Filtration						
Capital ($)	1,590,000	–	360,000	230,000	380,000	2.56
Ann. Cap & Replacement ($/Yr)	146,500	9,300	35,200	22,800	37,300	0.25
O&M ($/Yr)	223,000	72,300	40,100	30,300	42,900	0.41
Total Annual ($/Yr)	369,500	81,600	75,300	53,100	80,200	0.66
Haul @ $1/wet ton						
Annual ($/Yr)	36,500	8,600	3,540	2,500	3,900	0.06
Landfill						
Capital ($)	425,000	166,000	100,000	86,000	110,000	0.89
Annual Cap. & Replacement ($/Yr)	35,600	13,900	8,400	7,200	9,200	0.07
O&M ($/Yr)	116,800	49,750	28,700	23,900	30,550	0.25
Total Annual ($/Yr)	152,400	63,600	37,100	31,100	39,700	0.32
Grand Total ($/Yr)	601,400	179,600	137,200	105,900	177,100	$\boxed{1.20}$
Present Worth ($ Million)	9.32	2.78	2.13	1.64	2.74	$\boxed{18.6}$

[1] 6-1/8% @ 50 years

[2] Includes Owego Village

TABLE VI-13

INCINERATION COSTS – ALTERNATIVE A[1]

PC/AWT

	B-JC	Endicott	E. Owego	W. Owego[2]	Chenango Valley	Total ($ Million)
Thickener						
Capital ($)	290,000	213,000	83,700	76,000	83,700	0.75
Annual Cap. & Replacement ($/Yr)	31,700	20,600	8,800	8,200	9,000	0.08
O&M ($Yr)	4,400	1,900	1,500	1,800	2,050	0.01
Total Annual ($/Yr)	36,100	22,500	10,300	10,000	11,000	0.09
Vacuum Filtration						
Capital ($)	2.47×10^6	–	531,400	436,500	550,900	3.99
Ann. Cap. & Replacement ($/Yr)	220,000	12,900	44,500	36,700	46,200	0.36
O&M ($/Yr)	324,000	82,900	41,600	37,000	52,500	0.54
Total Annual ($/Yr)	544,000	95,800	86,100	73,700	98,700	0.90
Incineration						
Capital ($)	2.46×10^6	1.0×10^6	669,000	610,000	680,000	5.42
Ann. Cap. & Replacement ($/Yr)	207,000	84,400	55,700	50,700	57,300	0.46
O&M ($/Yr)	156,100	54,500	33,600	30,400	35,800	0.31
Total Annual ($/Yr)	363,100	138,900	89,300	81,100	93,100	0.77
Grand Total ($/Yr)	943,200	257,200	185,700	164,800	202,800	$\boxed{1.75}$
Present Worth ($ Million)	14.61	3.98	2.88	2.55	3.14	$\boxed{27.2}$

[1] 6-1/8% @ 50 years

[2] Includes Owego Village

TABLE VI-14

LAND APPLICATION COSTS – ALTERNATIVE B[1]

PC/AWT

	B-JC	Endicott	E. Owego	W. Owego[2]	Chenango Valley	Total ($ Million)
Thickener						
Capital ($)	290,000	213,000	83,700	76,000	83,700	0.75
Ann. Cap. & Replacement ($/Yr)	31,700	20,100	8,800	8,200	9,000	0.08
O&M ($/Yr)	4,400	1,900	1,500	1,800	2,050	0.01
Total Annual ($/Yr)	36,100	22,000	10,300	10,000	11,000	0.09
Haul @ $32/ton ($/Yr)	441,000	87,200	33,800	23,800	37,600	0.62
Total Annual ($/Yr)	477,100	109,200	44,100	33,800	48,600	$\boxed{0.71}$
Present Worth ($ Million)	7.39	1.70	0.68	0.52	0.75	$\boxed{11.0}$

[1] 6-1/8% @ 50 years

[2] Includes Owego Village

TABLE VI-15

LANDFILL COSTS – ALTERNATIVE C[1]

P/C AWT

	B-JC	Endicott	E. Owego	W. Owego[2]	Chenango Valley	Total ($ Million)
Thickener						
Capital ($)	290,000	213,000	83,700	76,000	83,700	0.75
Ann. Cap. & Replacement ($/Yr)	31,700	20,100	8,800	8,200	9,000	0.08
O&M ($/Yr)	4,400	1,900	1,500	1,800	2,050	0.01
Total Annual ($/Yr)	36,100	22,000	10,300	10,000	11,000	0.09
Vacuum Filtration						
Capital ($)	2.47×10^6	–	531,400	436,500	550,400	3.99
Ann. Cap. & Replacement ($/Yr)	220,000	12,900	44,500	36,700	46,200	0.36
O&M ($/Yr)	324,000	82,900	41,600	37,000	52,200	0.54
Total Annual ($/Yr)	544,000	95,800	86,100	73,700	98,400	0.90
Haul @ $1/wt Ton						
Annual ($/Yr)	55,200	10,910	4,200	3,000	4,700	0.08
Landfill						
Capital ($)	565,600	200,000	110,000	88,000	113,000	1.08
Ann. Cap. & Replacement ($/Yr)	47,400	16,800	9,100	7,500	9,500	0.09
O&M ($/Yr)	151,200	56,900	40,400	34,000	44,400	0.33
Total Annual ($/Yr)	198,600	73,700	49,500	41,500	53,900	0.42
Grand Total ($/yr)	833,900	202,400	150,100	128,200	168,000	1.48
Present Worth ($ Million)	12.92	3.14	2.33	1.99	2.60	23.0

[1] 6-1/8% @ 50 years

[2] Includes Owego Village

IMPACT ASSESSMENT

Tables VI-16 through VI-18 detail the quantifiable impacts associated with each sludge management alternative. The numbers represent the summation of all treatment plants in the Study Area. Where possible, comparisons were made to existing conditions. The following discussion is limited to the derivation of the numbers in the Tables.

The consumptive use of land for all three sludge handling alternatives was derived from the discussion given previously (Design Parameters), and does not include any buffering zones. This consumption of land was based on the 50-year planning period used in the Report. Land use for the landfill operation was considered as both a productive and consumptive use depending on the prior use of the land. The consumptive use of land for the land application alternative would be that required for the storage lagoon. The land required for disposal by land application was not considered a consumptive use; it was, however, included in the table for purposes of presentation of the actual land needed.

The particulate emission associated with truck travel was based on EPA exhaust emission standards of 1.2 gm/mile travelled. The emissions associated directly with the incinerator (particulate plus Hg) were determined from the EPA Process Design Manual for Sludge Treatment and Disposal. Under the resource consumption impacts, the amount of fuel was based on the assumptions stated previously (Design Parameters) for distances and gasoline consumption. Fuel would also be utilized in the incineration process for start-up, and if necessary, as an auxiliary fuel supply.

The latter requirement results when either the moisture content is high and/or the volatile content is low. Since for all treatment alternatives, sludge was assumed to possess a moisture content of 75 percent, the former factor was disregarded. For start-up, the fuel requirements are 1.7 gal/ ton of solids (EPA Manual for Sludge Treatment and Disposal). Because secondary treatment generally has a volatile solids content of 65 percent, no auxiliary fuel would be needed. However, for both Bio AWT (50 percent volatile) and P/C AWT (43 percent volatile), auxiliary fuel would be necessary. This factor was obtained from the EPA manual on sludge handling, which determines the amount of natural gas required in CF/ton of dry solids. This value was then converted into gallons of fuel oil.

TABLE VI-16

SECONDARY TREATMENT SLUDGE MANAGEMENT IMPACTS (YEAR 2020)

IMPACT	INCINERATION	LAND APPLICATION	LANDFILL
1) Consumptive use of land, acres (50 yr. period)	14	7.2	226
Non-consumptive use of land, acres (retrievable)		1770	
	Comparison: 800,000 total acres in Broome – Tioga Counties, ref: NYS Statistical Yearbook, 1974, (NYSSY)		
2) Air impacts:			
Particulate Emissions (lbs/yr)	17,425 (incinerator plus trucks)	230 (trucks)	190 (trucks)
	Comparison: Motor Vehicles in area emit 160,000 lbs/yr, ref: NYSSY WAPCA		
	11%	.14%	.12%
	Comparison: Power plant emits 15×10^8 lbs/yr		
Hg (lbs/yr)	.13	–	–
	Comparison: Coal burning in U.S. emits 3,000 tons/d ref: EPA		
3) Resource Consumption:			
Fuel (gallon/yr)	24,000 irretrievable	14,700 irretrievable	12,100 irretrievable
	Comparison: above consumption ≤1 gal/motor vehicle in area. Ref: NYSSY		
Power (Kwh/yr)	1.9×10^6 irretrievable	–	1.12×10^6 irretrievable
	Comparison: Power consumption in study area equals 1500×10^6 Kwh/yr		
	.12%		.07%
Dewatering Chemicals			
Lime (tons/yr)	1,067		830
$FeCl_3$ (tons/yr)	266		276
Lime for fertilizer & pH control (tons/yr)	retrievable	1,100 irretrievable	irretrievable
	Comparison: Lime consumption in area for agricultural use equals 107,000 tons/yr Ref: NYSSY		
	1%	1%	.8%

TABLE VI-16
(continued)

IMPACT	INCINERATION	LAND APPLICATION	LANDFILL
Nutrients (tons/yr)			
Nitrogen	270	–	140
Phosphorus	135	–	70
Potassium	26	–	9
	irretrievable		irretrievable
	Comparison: N-P-K added to farmland in Broome–Tioga Counties equal to 3145 tons/yr N, 1020 tons/yr P, 1530 tons/yr K ref: J. Agr. Food Chem.		
4) Resource Production			
Ash (tons/yr)	4,700 no profitable use at present	–	–
Digester gas (Cf/yr)	–	9.45×10^7 no auxiliary fuel required for digester heating at STP	9.45×10^7
Nutrients			
Nitrogen	–	185	–
Phosphorus	–	90	–
Potassium	–	18	–
	Comparison: N-P-K added to farmland equals 3,145, 1,020, 1,530		
5) Noise			
# truck trips/yr	467	7,300	3,536
	Comparison: number of truck trips for Solid Waste Disposal equals 31,000/yr. Ref: Municipal Refuse Collection and Disposal		
	1.5%	23%	11%
6) Return of scrubber water on STP (MGD)	1.2 Comparison: represents 3% of total plant flow		

TABLE VI-17

BIOLOGICAL AWT SLUDGE MANAGEMENT IMPACTS (YEAR 2020)

IMPACTS	INCINERATION	LAND APPLICATION	LANDFILL
1) Consumptive use of land (acres) (50 yr. period)	27	16	360
Non-consumptive use of land, acres (retrievable)		2765	
	Comparison: 800,000 total acres in Broome-Tioga Co.		
2) Air Impacts			
Particulate Emissions (lbs/yr)	24,850	480	275
	Comparison: Motor Vehicles in area emit 160,000 lbs/yr		
	15.5%	.3%	.17%
	Comparison: power plant emits 15 $\times 10^8$ lbs/yr		
Hg (lbs/yr)	.19	–	–
	Comparison: coal burning in US discharges 3,000 tons/day		
3) Resource Consumption			
Fuel (gal/yr)	330,000 irretrievable	184,000 irretrievable	18,000 irretrievable
	Comparison: above consumption <2 gal/motor vehicle in area.		
Power (Kwh/yr)	3.0×10^6 irretrievable	–	1.63×10^6 irretrievable
	Comparison: Power consumption in area equals $1,500 \times 10^6$ Kwh/yr		
	.2%		.1%
Dewatering chemicals			
Lime (tons/yr)	1,527	–	1,241
$FeCl_3$ (tons/yr)	381		414
Lime for fertilizer and pH control	irretrievable	1730 irretrievable	irretrievable
	Comparison: Lime consumption in area for agricultural use equals 107,000 tons/yr		
	1.4%	1.6%	1.2%
Nutrients (tons/yr)			
Nitrogen	315	–	180
Phosphorus	180	–	135
Potassium	31	–	11
	irretrievable		irretrievable
	Comparison: N-P-K added to farmland in Broome–Tioga Co. equals 3,145 tons/yr N, 1,020 tons/yr P, 1,530 tons/yr K.		

TABLE VI-17
(continued)

IMPACT	INCINERATION	LAND APPLICATION	LANDFILL
4) Resource Production Ash (tons/yr)	8,960 no profitable use at present	–	–
Digester gas	–	9.45×10^7 no auxiliary fuel required for digester heating at STP	9.45×10^7
Nutrients (tons/yr)	–		–
Nitrogen		240	
Phosphorus		180	
Potassium		22	
		Comparison: N-P-K added to farmland equal 3,195, 1,020, 1,530	
5) Noise # truck trips/yr	895	15,300	5,688
	Comparison: number of truck trips for solid waste disposal equals 31,000/yr		
	2.9%	50%	18%
6) Return of Scrubber Water on STP (MGD)	1.7 represents 4% of total plant flow	–	–

TABLE VI-18

PHYSICAL/CHEMICAL AWT SLUDGE MANAGEMENT IMPACTS (YEAR 2020)

IMPACT	INCINERATION	LAND APPLICATION	LAND FILL
1) Consumptive use of land (acres) (50 yr. period)	30	16	425
Non-consumptive use of land, acres (retrievable)		2825	
	Comparison: 800,000 acres total in B-T Co.		
2) Air Impacts particulate emission (lb/yr)	25,350	485	295
	Comparison: Motor Vehicles in area emit 160,000 lbs/yr		
	15.8%	.3%	.18%
	Comparison power plant emits 15 × 10^8 lbs/yr		
Hg (lbs/yr)	19	–	–
	Comparison: coal burning in U.S. discharges 3,000 tons/day		
3) Resource Consumption Fuel (gal/yr)	670,000 irretrievable	31,000 irretrievable	18,600 irretrievable
	Comparison: above consumption <5 gal/motor vehicle		
Power (Kwh/yr)	3.1 × 10^6 irretrievable	–	2.15 × 10^6 irretrievable
	Comparison: Power consumption in area equals 1,500 × 10^6 Kwh/yr		
	.2%		14%
Dewatering Chemicals Lime (tons/yr)	1,558	–	1,558
$FeCl_3$ (tons/yrs)	388		388
Lime for fertilizer and pH control	irretrievable	1,765 irretrievable	irretrievable
	Comparison: Lime consumption in area for agricultural use equals 107,000 tons/yr		
	1.5%	1.6%	1.5%
Nutrient (tons/yr) Nitrogen	320		320
Phosphorus	180		180
Potassium	31 irretrievable		31 irretrievable
	Comparison: N-P-K added to farmland in B-T Co equals 3,145 tons/yr N, 1,020 tons/yr P. 1,530 tons/yr K.		

TABLE VI-18

(Continued)

IMPACT	INCINERATION	LAND APPLICATION	LAND FILL
4) Resource Production	10,000		
Ash (tons/yr)	no profitable use at present	–	–
Nutrients (tons/yr)			
Nitrogen	–	240	–
Phosphorus	–	180	–
Potassium	–	22	–
	Comparison: N-P-K applied to farmland in area equals 3,145, 1,020, 1,530		
5) Noise			
# truck trips/yr	1,003	15,500	5,652
	Comparison: number of truck trips for solid waste disposal equals 31,000		
	3.2%	50%	18%
6) Return of scrubber water On STP (MGD)	1.8 MGD represents 4.5% of total plant flow		

Power would be used to operate both the vacuum filter and incinerator. The incinerator power consumption was based on a requirement of 50 KWH/ton of dry solids. The power consumption for vacuum filtration assumed the cost associated with power was 8 percent of the O&M costs (EPA manual for Sludge Treatment), and a cost per KWH of 2 cents.

The lime and ferric chloride utilized for sludge conditioning in the vacuum filtration process was derived from the WPCF design manual (Sewage Treatment Plant Design). For digested sludge the amount of lime and $FeCl_3$, required would be 9 percent and 3 percent, respectively, of the dry weight of sludge, and for undigested sludge 8 percent and 2 percent, respectively. The quantity of lime used for the land application alternative was based on 0.6 tons/ac/year.

The quantities of nutrients, either consumed or produced, were computed using the ratios stated previously in Design Parameters.

Regarding resource production, methane gas generated by the digestion process was calculated on the assumption that 1 CF/cap/day would be produced (Sewage Treatment Plant Design, WPCF). Ash produced from the incineration process was assumed to be equivalent to the non-volatile content of the sludge at 100 percent solids. Ash may be utilized productively in making building bricks or in stabilizing highway and road subgrades, but neither use is a profitable one at present (Culp and Culp, 1971).

The noise impact, associated with truck trips, was determined from the assumptions stated in Design Parameters (10-ton dump trucks or 6,000 gallon tanker trucks and distance to landfill site or land application site).

The return of scrubber water would have a direct impact on the STP because this flow would be equal to 400 times the dry solids flow rate to the incinerator (EPA manual for Sludge Treatment).

IMPACT EVALUATION

This section summarizes the impacts of the sludge handling alternatives on land, air, water, resource consumption, and resource production (see Impact Assessment and Evaluation Appendix for details). The amount of consumptive land (land committed over the project life and precluded from other uses) and non-consumptive land (continuance of existing land use) required for each alternative is given in Table VI-19.

TABLE VI-19

SLUDGE MANAGEMENT LAND REQUIREMENTS
(Based on 50-Year Project Life)(Acres)

Impact	A Incineration	B -- Land Application	C Landfill
Secondary Treatment			
Consumptive Use*	14	7	226
Non-Consumptive Use**	0	1,771	0
Biological AWT			
Consumptive Use	27	16	360
Non-Consumptive Use	0	2,765	0
Physical/Chemical AWT			
Consumptive Use	30	16	425
Non-Consumptive Use	0	2,825	0

*Consumptive Use--purchased and irretrievable.
**Non-Consumptive Use--not purchased; existing agricultural land used.

INCINERATION

Incineration would have adverse environmental impacts on the air quality and resource consumption categories. The particulates emitted by the incinerator, although significant in comparison to the motor vehicle traffic, would lose their importance in comparison to the power plant emissions. However, the Binghamton area has been designated an air quality maintenance area with the primary concern being suspended particulates. In light of this designation, any

additional point sources of particulate emissions must not be taken lightly. The incineration process would also consume a considerable amount of natural resources including fuel, electricity, nutrients, and chemicals. These values would lose their significance in comparison with the total consumption of these resources in the Study Area; however, in terms of environmental conservation, extensive use of these resources should be prevented if possible. Incineration would provide no beneficial impacts and would produce no significant adverse impacts concerning land and water.

LAND APPLICATION

The beneficial impacts associated with land application would be the nonconsumptive use of land and an improvement of this land by soil conditioning. However, 7 to 16 acres would be committed over the life of the project to provide storage during nonapplication periods. This commitment would be minor in comparison with the amount of irretrievable land used for landfill of dewatered sludge and would be one-half that used for disposal of incinerated ash.

Resource consumption would be minimized by eliminating the vacuum filter operation, and by the fertilizing effect of the nutrients from the sludge being recycled on farmland rather than wasted. The organic matter in the sludge would act as a soil conditioner by building up the humus material in the soil. The other major impact categories involved, air and water, would be unaffected by land application.

LANDFILL

Landfill would also not provide any beneficial impacts and would have an adverse effect on resource consumption. This would be as a result of the vacuum filter operation and a nonproductive use of the sludge itself. By hauling to a landfill, the nutritional value of the sludge would be lost. Landfill would not significantly affect the impacts regarding land, air or water, and so would be a slightly better alternative than incineration.

RECOMMENDATIONS

On an impact evaluation basis, land application was the clear choice for a sludge handling and disposal method, regardless of the level of treatment. Likewise, land application was also the chosen method derived from a cost-effective analysis. The farmland for sludge disposal is readily available (170,000 acres of agricultural land in Broome and Tioga Counties) and land application would, in turn, benefit the farmer (nutrients and organic content present in the sludge). However, the above considerations are given with certain reservations.
There does exist the potential for heavy metals build-up in the soils and/or toxicity effects of the metals in plants. This may particularly apply to sludge from the Endicott STP which has been experiencing recent problems with heavy metals. Since work is currently being conducted on controlling this problem in the treatment plant, it was assumed this would not be the case in the future. Nitrate pollution of groundwater as a result of nitrification and subsequent leaching is also possible.

These two possibilities could be mitigated by selecting a correct application rate and strict monitoring at the site. The application rate utilized in the design (5 tons/acre/year) was based on the nitrogen uptake rate of particular crops (corn, grains) and should, therefore, prevent nitrogen pollution from occurring. Strict monitoring of the soil and crops for heavy metals build-up would mitigate heavy metal toxicity effects. In other words, monitoring would determine what the short term and long term effects are, if any, on the soil and crops. Modifications to the land application system could then be made (such as a change in application rate) if monitoring indicated a potential problem.

A potential odor problem could exist at the storage site, especially regarding the P/C AWT systems (due to the undigested sludge). This impact could be mitigated in part by site location.

In summation, although potential impacts do exist for land application, these could easily be avoided or alleviated by the above mentioned suggestions.

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BINGHAMTON WASTEWATER MANAGEMENT STUDY. DESIGN AND COST APPENDI--ETC(U)
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The fact that little capital expenditure would be involved for the land application alternative indicates this method would be a reversible system: that is, if problems do arise which cannot be solved, another method of sludge handling could be implemented without much loss of money. This fact in itself is somewhat of a mitigating measure.

Because of the lower cost involved and the beneficial impacts associated with this method, the management committees for the Study recommended land application as the primary sludge disposal means for both Broome and Tioga Counties. Landfill of dewatered sludge was recommended as a back-up system should unavoidable problems arise with land application.

CHAPTER VII

STORMWATER MANAGEMENT

The present Study is concerned with the water quality management problems in the urban areas of Broome and Tioga Counties. Local government and planning boards of the two counties have indicated their concern for urban stormwater runoff problems in the region. In the following sections, the storm runoff problems and their water quality impacts are investigated. Feasible urban stormwater control alternatives are then discussed for the City of Binghamton.

STORMWATER RUNOFF PROBLEMS

During periods of storm inflow or high infiltration rates, the wastewater flows in the combined sewer system can exceed the capacities of the Binghamton sewer system and treatment plant with significant pollution loads discharged to the Susquehanna River. This would render wastewater management alternatives that are based only on dry weather conditions ineffective in achieving water quality objectives during storm conditions.

A recent drainage and stormwater runoff study (Shumaker and Hawk, 1974) has been performed for the Southern Tier East Regional Planning Board. Existing major drainage and stormwater problems throughout the area were catalogued and future problems under project conditions were discussed.

The two counties were divided into planning basins. The major drainage problems of the City of Binghamton planning basin were found to include erosion along both large and small creeks and problems of overflows associated with the existing combined storm and sanitary sewer system. This basin was recommended for immediate master planning because of the existing and anticipated future storm drainage problems.

In the Endicott area, the major problems were channel erosion in the Village of Endicott along Brixius Creek and roadway flooding on Country Club Road. It was recommended that efforts should be directed toward systematic maintenance of existing facilities. Among these facilities are two Soil Conservation Service dams protecting most of the Endicott area from flooding. It was also recommended that steps be taken to preserve flood plains through proper planning and zoning actions.

For the planning basin that included the Village of Owego, it was found that the existing drainage problems include flooding of Owego Creek in the Village. Future problems in storm drainage were found to be likely along Owego Creek unless preventive measures are taken in upstream planning basins or additional corrective measures are taken in the downstream areas. Local problems were found to be probable because of the projected future urbanization for all sides of the Village of Owego according to the General Plan. It was recommended that efforts be made toward further stabilization of the Owego Creek channel and systematic maintenance of existing drainage facilities. Also, it was recommended that new development be prohibited from encroaching on flood plains.

Alternative methods for the correction of drainage and stormwater problems were reviewed. These methods included the reconditioning or replacement of unsatisfactory drainage facilities, the enlargement of existing systems, provision of additional drainage systems, multiple use of existing upstream storage sites, and land use controls. It was concluded the optimum solution for drainage problems would consist of an economic choice between the different combinations of the above systems. To achieve an economical and effective solution, planning on the regional level was recommended.

The Shumaker & Hawk study recommended the preparation of a Criteria Manual for the orderly and consistent design of storm drainage systems and the detailed master planning of selected basins on a priority list. The possibility of multiple use of county regional park sites as stormwater retention facilities was highlighted. The Watershed Protection Plan (Public Law 566) provided for the most flood protection at the least cost to local residents.

As mentioned, serious water quality problems could result from storm runoff in areas served by combined sewer systems. Within the Study Area, a portion of the Binghamton-Johnson City area and the Village of Owego are served by

combined sewers. The Endicott area is mainly served by separate sanitary and storm sewers as is the Town of Owego. Examination of the sewer maps showed about 85 percent of the total area is served by separate storm and sanitary sewers. Table VII-1 presents the precentage of combined and separate sewers in the existing service areas.

A study had also been prepared for the Village of Owego (K. G. Woodward and Associates, 1968) recommending the separation of storm and sanitary sewer systems. New York State has requested that work be initiated on separation of combined sewer systems. For this reason, and because the consulting firm of Shumaker Engineers was performing a concurrent sewer system analysis for the City of Binghamton, most of the effort for stormwater management was expended in developing control measures for the Binghamton area. Later in the Study, some methods of stormwater management developed for the City of Binghamton were also applied in the Village of Owego to help solve its overflow problems.

TABLE VII-1

EXISTING SEWERAGE SYSTEMS

Service Area	Type of Sewers*	Population Served
Binghamton-Johnson City	Combined--70% Separate--30%	102,000
Vestal	Separate--100%	7,500
Endicott	Combined--15% Separate--85%	44,700
Owego STP No. 1	Separate--100%	600
Owego STP No. 2	Separate--1')%	6,500
Owego Village	Combined--65% Separate--35%	3,600
Owego Valley View	Separate--100%	500
TOTAL STUDY AREA		165,400

*Based on Percent of Land Area

Existing and future wastewater flows from the Owego Village area are significantly less than those from the Binghamton-Johnson City area. The existing average wastewater flow in the former area is less than 1 mgd as compared to 18.25 mgd for the latter area. Thus, it is expected the storm overflow pollution load to the Susquehanna River that could occur in the Binghamton-Johnson City area is much greater than the corresponding pollution load from the Owego Village area.

STORM RUNOFF AND COMBINED OVERFLOWS IN THE CITY OF BINGHAMTON

Conferences with local governments and planning boards of Broome and Tioga Counties, NYSDEC, and EPA officials indicated storm runoff and combined overflows from the City of Binghamton are a problem of major concern. The area has the largest population in the Bicounty Area and is served by a relatively old combined sewer system. The recently completed secondary wastewater treatment facility for the Binghamton-Johnson City area was planned to be a regional facility to handle wastewater flows from surrounding urban areas. Due to infiltration/inflow problems, the plant may not be capable of serving these adjacent areas. The major trunk and interceptors of the existing sewer system in the City of Binghamton are shown on Plate 1. The Binghamton sewer system was found to be subjected to significantly higher flow rates during wet weather conditions. The combined sewer system overflows to the Chenango and Susquehanna Rivers during these excessive flows, resulting in the discharge of raw sewage to the waterways. The purpose of the Shumaker study (1974) mentioned earlier was to define the sources of these higher flows and to outline possible correctional measures. The Village of Johnson City has also been directed to undertake an analysis and evaluation of its sewer system.

INFILTRATION/INFLOW PRELIMINARY ANALYSIS

Preliminary analysis of urban runoff inflow and infiltration was performed using the Binghamton-Johnson City wastewater treatment plant continuous flow and quality records,

rainfall data, and Susquehanna River stage data. Empirical equations were used to calculate approximately the surface runoff volume and combined overflow magnitude resulting from a given storm. Later, a more detailed analysis of storm runoff and sewer overflow volumes and pollutant magnitudes was performed using a mathematical simulation model.

In the preliminary work, existing wastewater flow data recorded by the Binghamton-Johnson City sewage treatment plant were analyzed to test the existence of infiltration or inflows. Rainfall records and Susquehanna River flow data were examined to test the correlation between treatment plant flows and precipitation or high river stages. Treatment plant influent water quality records were analyzed to check the variation of pollutant mass loads reaching the treatment plant during various flow rates.

Infiltration

The Binghamton-Johnson City wastewater treatment plant continuous flow record was analyzed for the year 1973. During the period of August-October, the daily river stage data indicated the river was at its lower stages. Investigation of the continuous treatment plant flow record indicated that dry weather flow during the period of low river stages was above 15 mgd (during the day the flow varied between 7 mgd during early morning hours to about 20 mgd around noon time). Sanitary flow, based on water consumption and population, was estimated at 11 mgd. This indicated that at least 4 mgd reaching the treatment plant during dry weather and low river flow conditions was due to infiltration. Infiltration during these months was thought to be significantly smaller than during high river stage periods. Higher river stages would cause higher groundwater levels in the vicinity of the Susquehanna and Chenango Rivers where the interceptors are located. This probably causes an increase in infiltration rates. Also, increases in river stage could cause direct river water infiltration to the sewer system through faulty regulators or broken parts of the sewer system that might otherwise be above river elevation during lower summer flows.

The treatment plant data for 1973 indicated excessive plant flows for July and December. The increase in treatment plant wastewater flow above the dry weather flow was plotted against river stage (Figure VII-1). An apparent correlation existed between plant flow and river stages.

BINGHAMTON–JOHNSON CITY
EXCESS TREATMENT PLANT FLOW VS. RIVER STAGE
(DRY WEATHER CONDITIONS)

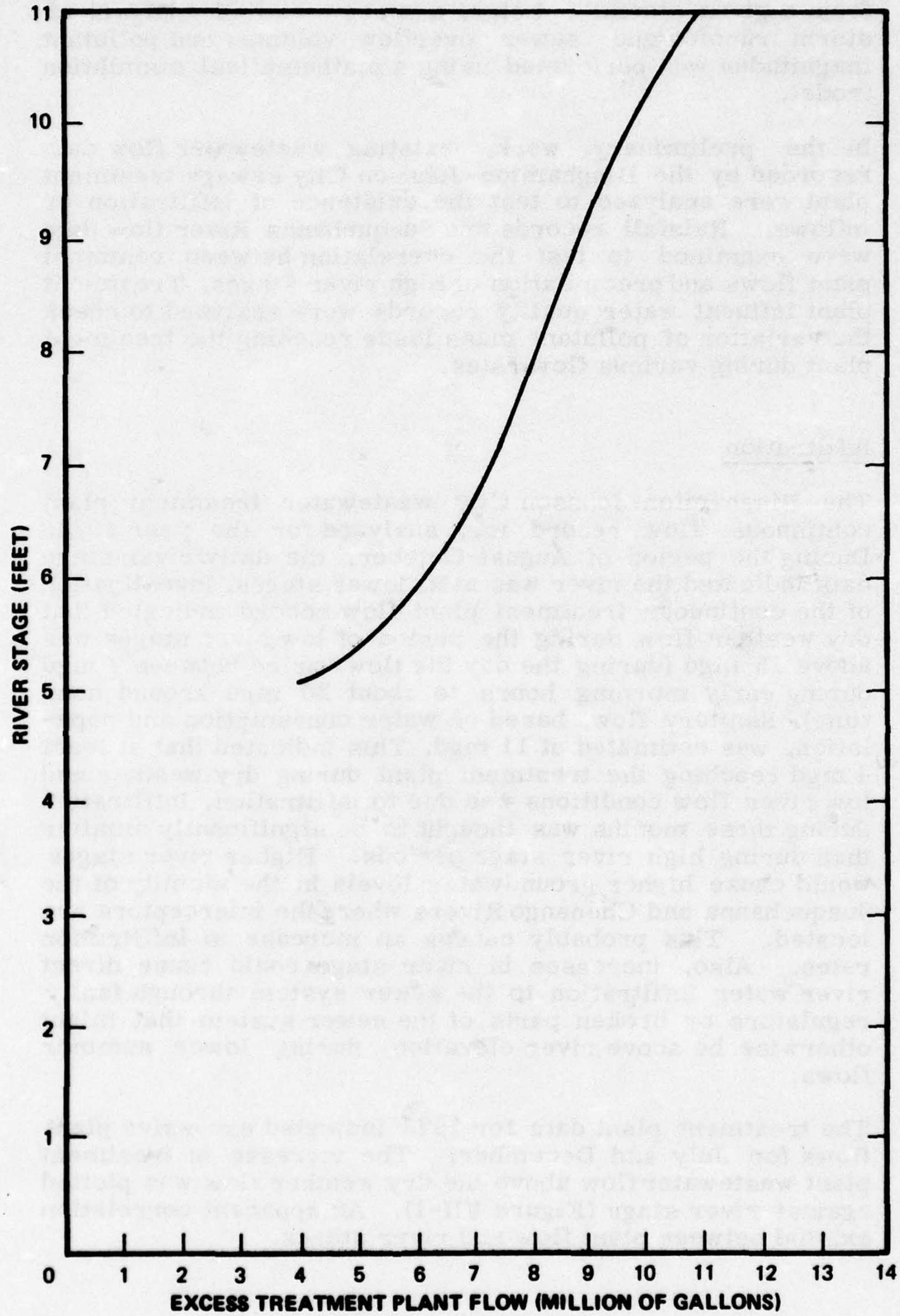


FIGURE VII-1

Inflow

To estimate storm inflow, 14 storms occurring in the summer and fall of 1973 were analyzed. The hourly and daily rainfall data as reported by the US Weather Bureau at Broome County Airport were utilized to estimate the total volume of rainfall over the area. Treatment plant flow records were studied to determine the increase in treatment plant flow due to the rainfall. A summary of plant flows and rainfall data of these 14 storms occurring during low river stages is given in Table VII-2. Based on previous studies of urban storm runoff, detention losses (0.1 inch for rainfall less than the 0.4 inches and 0.2 inches for rainfall greater than 0.4 inches) were subtracted from the total rainfall to determine "adjusted" rainfall.

The ratio of the urban runoff volume reaching the treatment plant to the "adjusted" rainfall volume was between 0.03 to 0.10. Experimental studies in other urban areas with combined sewers but no overflows indicated a ratio of between 0.2 to 0.5. The lower ratio for Binghamton is an indication of storm runoff discharged directly from the combined sewer system to the river before reaching the wastewater treatment plant.

From discussions with the STP operator, the treatment plant was found to receive up to 30 mgd due to storm inflow during heavy storm conditions. For a few hours during a rainy day, the flow rate was above 40 mgd. The correlation between rainfall and the corresponding increase in treatment plant flow is given in Figure VII-2.

Investigation of the influent data record of the treatment plant (January 1973 to March 1974) indicated the pollutant mass loadings (BOD, suspended solids) measured in lb/day decreased as the flow reaching the treatment plant increased (Figure VII-3). That is, overflows caused a portion of the daily sanitary load to escape to the river, thus decreasing the net pollutant load to the treatment plant. These direct discharges of raw sewage to the rivers caused a reduction of up to 50 percent in the pollutant load reaching the treatment plant. In other words, about 50 percent of the raw sewage load was reaching the natural streams during high sewer flows due to both stormwater and infiltration.

TABLE VII-2

BINGHAMTON-JOHNSON CITY TREATMENT PLANT AND RAINFALL-RUNOFF DURING LOW RIVER STAGES

Storm No.	Date (1973)	Increase in Treatment Plant Flow (mg)	Rain (Inches)	Adjusted Rainfall[1] Inches	Adjusted Rainfall[1] Million Gallons	Ratio of Increased Plant Flow to Adj. Rainfall	Rainfall Duration (hours)	Maximum Intensity Inch/hr
1	8/19	2.6	0.71	0.51	87.0	0.03	3	0.50
2	8/27	0.5	0.13	0.03	5.1	0.10	5	0.06
3	9/1	1.2	0.18	0.08	13.6	0.09	2	0.16
4	9/5	3.7	0.62	0.42	71.7	0.05	4	0.20
5	9/6	2.6	0.21	0.11	18.7	0.14	4	0.12
6	9/14	6.4	0.77	0.57	97.3	0.07	10	0.22
7	9/18	3.8	0.62	0.42	71.7	0.05	6	0.19
8	10/2	1.0	0.25	0.15	25.6	0.04	8	0.12
9	10/3	0.7	0.14	0.04	6.8	0.10	1	0.12
10	10/9	1.5	0.36	0.26	44.4	0.03	2	0.24
11	10/18	1.6	0.19	0.09	15.4	0.10	9	0.06
12	10/29	16.0	1.25	1.05	179.2	0.09	15	0.22
13	11/1	3.0	0.20	0.10	17.1	0.17	7	0.08
14	11/8	4.5	0.46	0.26	61.4	0.10	10	0.11

[1] Adjusted rainfall amount after subtracting losses to surface detention.

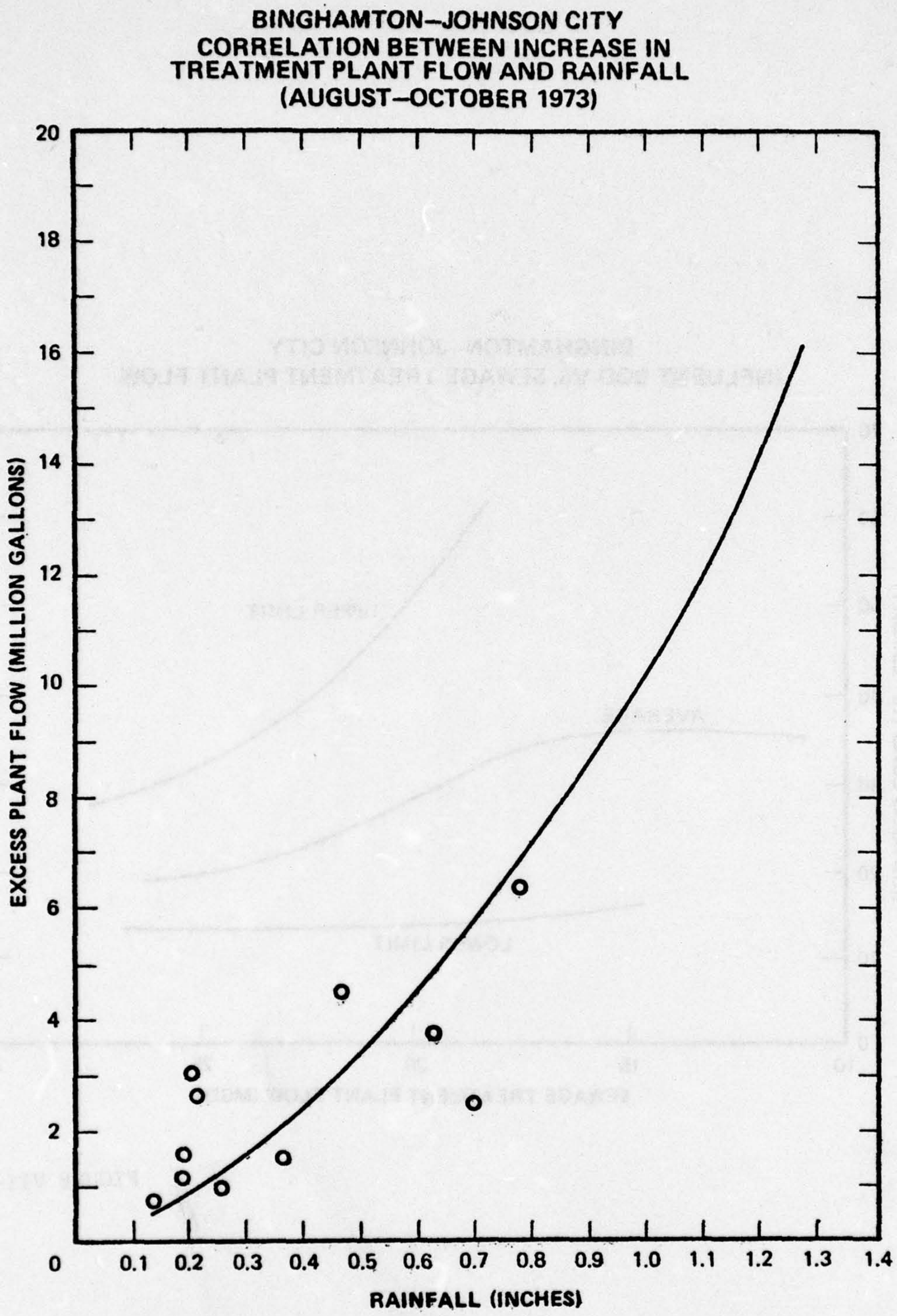

BINGHAMTON–JOHNSON CITY
CORRELATION BETWEEN INCREASE IN
TREATMENT PLANT FLOW AND RAINFALL
(AUGUST–OCTOBER 1973)
EXCESS PLANT FLOW (MILLION GALLONS)
20
18
16
14
12
10
8
6
4
2
0
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4
RAINFALL (INCHES)

FIGURE VII-2

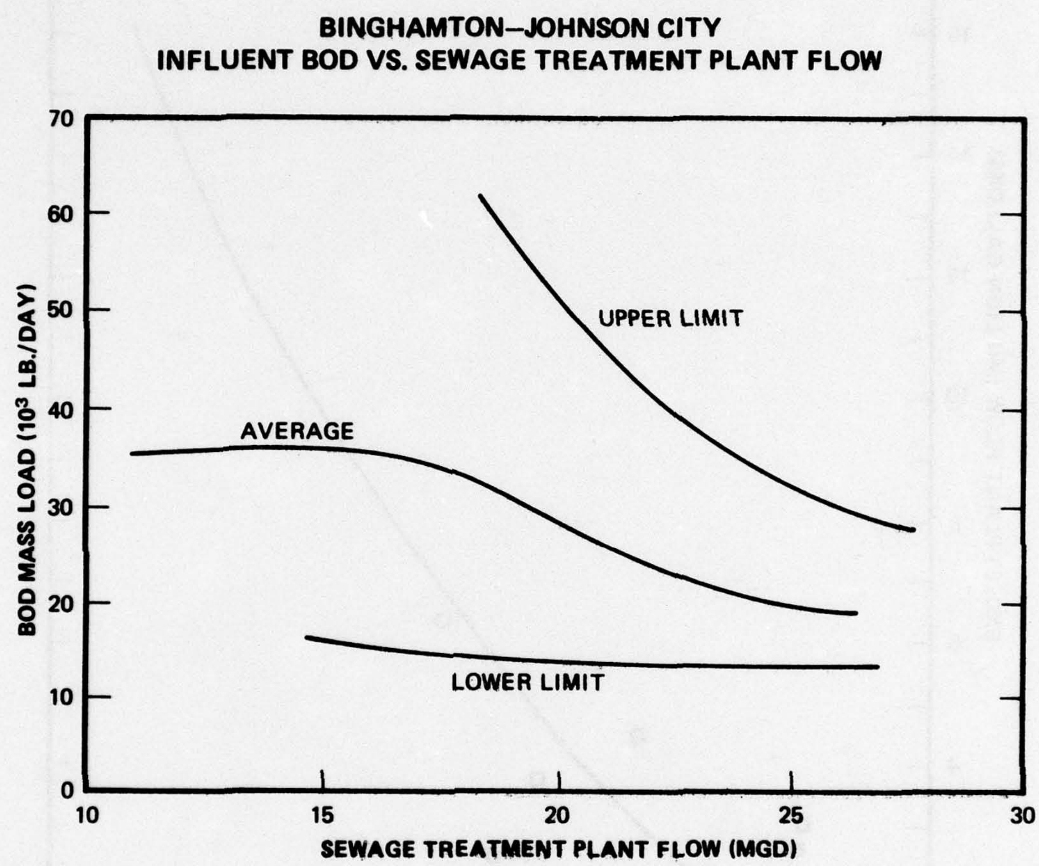

BINGHAMTON–JOHNSON CITY
INFLUENT BOD VS. SEWAGE TREATMENT PLANT FLOW
BOD MASS LOAD (10³ LB./DAY)
70
60
50
40
30
20
10
0
10
15
20
25
30
UPPER LIMIT
AVERAGE
LOWER LIMIT
SEWAGE TREATMENT PLANT FLOW (MGD)

FIGURE VII-3

CORRELATION BETWEEN COLIFORM AND RAINFALL

Previous experimental and analytical studies (Lager and Smith, 1973) have indicated that in a typical urban area served by secondary wastewater treatment facilities, the organic load of combined sewer overflows is of the same order of magnitude as the effluent load from a secondary STP. These studies have also shown the coliform content of combined overflows to be significantly higher than secondary effluent thereby causing serious water quality problems.

Existing Susquehanna River water quality data in the Binghamton area were examined to evaluate the impacts of the overflows for the summers of 1970 and 1973. The reported data indicated coliform counts increased significantly downstream of the City following rainfall occurrences. In order to isolate sewer overflow pollution from treatment plant effluent discharges, the water quality station at Mile Point 37.8 was selected for further analysis. This station is just upstream of the Binghamton-Johnson City Sewage Treatment Plant and is, therefore, not affected by the effluent from any of the treatment plants but is subject to the effects of combined sewer overflows.

Statistical analysis for the summer periods of 1970 to 1973 indicated correlations between coliform count and rainfall intensity and between coliform count and total rainfall (a less significant correlation than the former).

Linear regression techniques were used to fit the curve shown in Figure VII-4. The t-Test was performed to prove the hypothesis of zero correlation. It was found that this hypothesis would be rejected at the 5 percent level of significance.

The relationship between coliform and rainfall shown in Figure VII-4 shows that, following higher intensity rainfalls, coliform counts for the Susquehanna River are significantly greater than values allowed by New York State standards for all stream classifications (2,400 for Class B, 5,000 for Class A; and 10,000 for Class C). Investigation of coliform counts reported at water quality stations upstream of the City of Binghamton indicates bacterial concentrations are always an order of magnitude less than those below the City during storm periods. Previous studies in other areas have indicated that the coliform count of storm runoff is an order of magnitude lower than for combined overflows

CORRELATION BETWEEN COLIFORM COUNT AND RAINFALL INTENSITY (SUSQUEHANNA RIVER MILE POINT 37.8, 1970–1973)

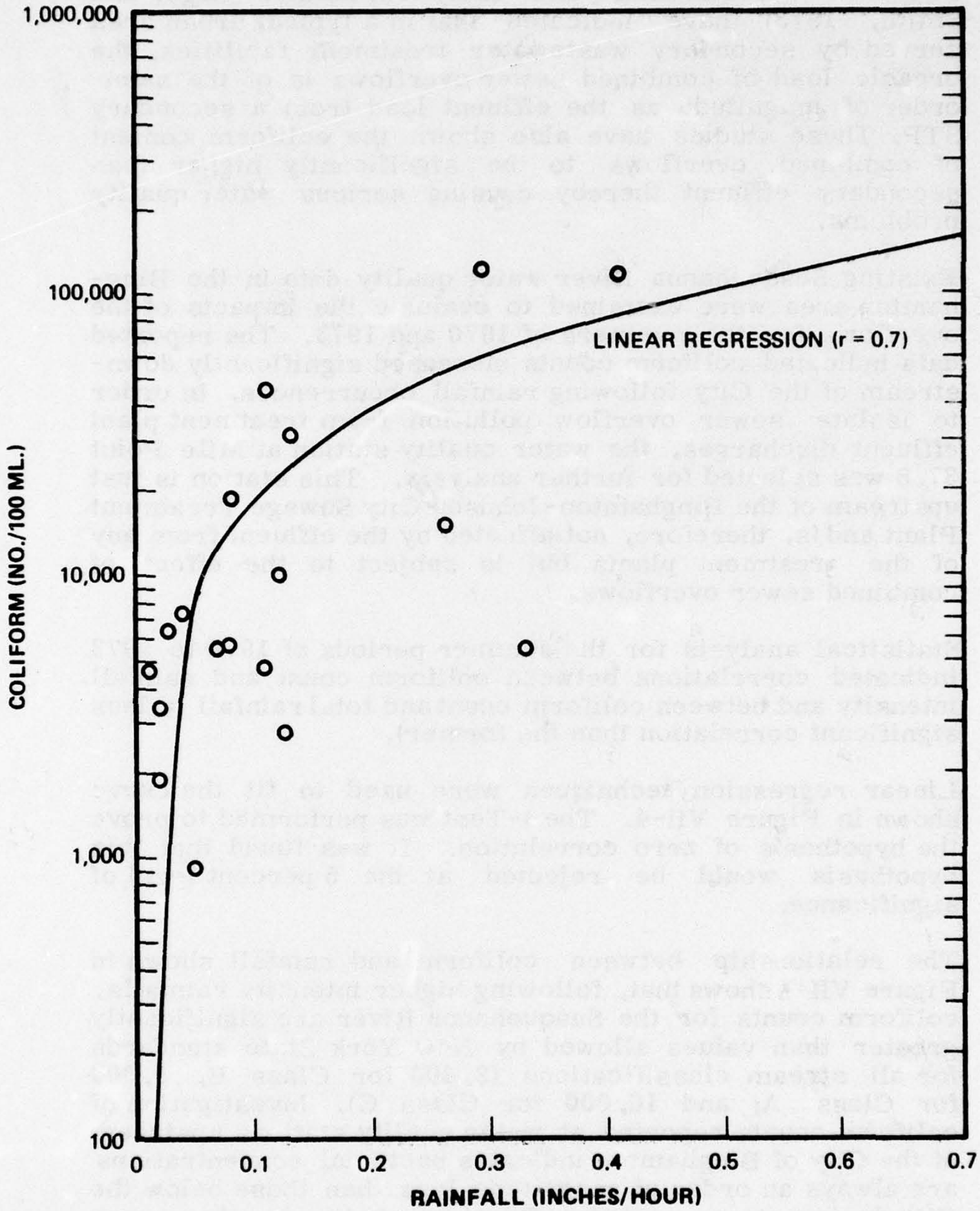


FIGURE VII-4

(Lager & Smith, 1974). Water quality samples of combined sewer overflows in the Binghamton area have indeed shown high coliform discharges (Following section). Hence, these higher coliform levels in the Susquehanna River were attributed to the combined overflows in the Binghamton area.

MONITORING OF STORM OVERFLOWS

Monitoring of pollutant discharges from overflows and by-passes was essential in assessing the impact of these over-flows on the receiving streams and in evaluating the alter-native measures to abate pollutants from this source.

Conferences were held with the City of Binghamton Engi-neering Department and the Broome County Health Department to investigate the feasibility of conducting such a monitoring program. The City of Binghamton agreed to provide for the measurement of overflow rates while the Broome County Health Department would provide for the water quality analyses of the samples collected from the overflow sites.

Discussion with the engineers conducting the infiltration/inflow study for the City of Binghamton (Shumaker, 1975), indicated five sites offering the most practical locations for obtaining flow rates and wastewater samples for analysis. They were (1) Fourth Ward Trunk and Northside Interceptor, (2) Laurel Avenue Trunk and Charles Place, (3) Murray Street and Northside Interceptor, (4) Conklin Avenue and South Washington Street, and (5) Court Street and Tompkins Street.

Results of the overflow quantity and quality measurements conducted in the summer of 1974 at three of the sites are summarized in Tables VII-3 through VII-6. Analysis of these results provided a preliminary evaluation of the water quality problems that could result from storm overflows. The biochemical oxygen demand (BOD) of the overflows was in the range of 24 to 96 mg/l with an average of 59 mg/l, which may be compared to the effluent BOD value from the Binghamton-Johnson City STP of about 20 mg/l. A much more serious water quality problem was indicated by the fecal coliform count; the range reported was between 240,000 MPN/100 ml to 2,400,000 MPN/100 ml. It was noted that overflow quality exhibited significant differences depending on the storm conditions and the location of the overflow.

TABLE VII-3
COMBINED SEWER OVERFLOWS
FOURTH WARD TRUNK AND NORTHSIDE INTERCEPTOR

Location: 4th Ward Trunk

Date: 3 September 1974

Rainfall: 0.49" between 1 a.m. and 9 a.m. (Broome County Airport)

Time of Overflow Onset: 1:00+ Sampling: 1:30

	1st 15 Minutes	2nd 15 Minutes
Flow (cfs)	1.0	1.0
Total Solids (mg/l)	157	158
Suspended Solids (mg/1)	65	50
Nitrate as N (mg/1)	0.94	1.0
BOD_5 (mg/1)	40	37
Chlorides (mg/1)	16	16
Nitrogen, Kjeldahl (mg/1) includes ammonia	6.1	4.4
Coliform Bacteria (MPN/100 ML)	240,000	–
Fecal Coliform Bacteria (MPN/100 ML)	240,000	–

TABLE VII-4

COMBINED SEWER OVERFLOWS
LAUREL AVENUE TRUNK AND CHARLES PLACE

Location: Laurel Avenue

Date: 3 September 1974

Rainfall: 0.31″ between 1 a.m. and 7 a.m. (Broome County Airport)

Time of Overflow Onset: 11:30 Sampling: 11:50

	1st 15 Minutes
Flows (cfs)	0.6
Total Solids (mg/l)	124
Suspended Solids (mg/1)	37
Nitrate as N (mg/1)	0.82
BOD_5 (mg/1)	24
Chlorides (mg/1)	14
Nitrogen, Kjeldahl (mg/1) includes ammonia	5.0
Coliform Bacteria (MPN/100 ML)	240,000
Fecal Coliform Bacteria (MPN/100 ML)	240,000

TABLE VII-5

COMBINED SEWER OVERFLOWS
MURRAY STREET AND NORTHSIDE INTERCEPTOR

Location: Murray Street

Date: 15 October 1974

Rainfall: 0.2" between 8:00 a.m. and 9:00 a.m.

Time of Overflow Onset: 8:30 a.m. Sampling: 8:45

	1st 15 Minutes	2nd 15 Minutes	3rd 15 Minutes
Flow (cfs)	5.2	3.1	0.6
Total Solids (mg/l)	441	333	346
Suspended Solids (mg/l)	184	104	56
Nitrate as N (mg/l)	0.1	0.1	0.1
BOD_5 (mg/l)	96	67	76
Chlorides (mg/l)	16	13	18
Nitrogen, Kjeldahl (mg/l) includes ammonia	12.3	10.1	10.9
Coliform Bacteria (MPN/100 ML)	2,400,000	2,400,000	2,400,000
Fecal Coliform Bacteria (MPN/100 ML)	2,400,000	430,000	1,100,000

TABLE VII-6

COMBINED SEWER OVERFLOWS
MURRAY STREET AND NORTHSIDE INTERCEPTOR

Location: Murray Street

Date: 16 October 1974

Rainfall: 0.2" between 5 and 9 a.m. and 0.2" between 9 and 3 p.m.

Time of Overflow Onset: 8:45 a.m. Sampling: 9:20

	1st 15 Minutes	2nd 15 Minutes	3rd 30 Minutes*
Flow (cfs)	1.4	0.2	0.4
Total Solids (mg/l)	261	303	276
Suspended Solids (mg/l)	48	58	60
Nitrate as N (mg/l)	0.4	0.46	0.40
BOD_5 (mg/l)	65	65	63
COD (mg/l)	240	190	160
Nitrogen, Kjeldahl (mg/l) includes ammonia	10.1	9.5	8.4
Coliform Bacteria (MPN/100 ML)	750,000	2,400,000	2,400,000
Fecal Coliform Bacteria (MPN/100 ML)	750,000	2,400,000	430,000

*Overflow stopped and then started about 3 hours later.

APPLICATION OF STORMWATER MODELS

The analysis of the storm overflow problems indicated the significance of overflow pollutant magnitudes. Because of the lack of direct measurements of urban runoff, infiltration in various sewer elements, and combined overflows, it was felt a comprehensive analysis utilizing mathematical and digital computer simulation models was warranted. The models could provide a more accurate estimate of stormwater quantity and overflow volume and pollutant magnitudes. Also, the models could be valuable in evaluating the effectiveness of alternative combined overflow management systems.

Two computer programs were utilized. The first was the Storm Water Management Model (SWMM) developed by the Environmental Protection Agency (Metcalf and Eddy, 1971). This water quality model was used to estimate overflow volume and pollutant (BOD & NOD) loading within the sewer system. In evaluating the impacts of overflows on the DO level of the river, it was necessary to use another model simulating the dynamic conditions of stormwater overflows within the river system. For this reason, Receive II, also developed by EPA, was used to estimate the effect of the pollutant loadings on the river.

As input to the SWMM, a frequency analysis was made to determine the appropriate design storm. The design river flow, based on yearly low flows, was computed for use in the Receive II model.

CORRELATION BETWEEN RAINFALL AND SUSQUEHANNA RIVER FLOW IN THE BINGHAMTON AREA

In estimating the long term probability of a storm overflow control measure for achieving its water quality objectives, it was necessary to consider the probability distribution of river flows given a rainfall of specific volume: in other words, test for a statistical correlation between rainfall and river flow. Daily rainfall records in the Binghamton area and average daily Susquehanna River flows recorded at Vestal were examined to determine if there was a correlation between these two sets of data.

Daily rainfall volume and Susquehanna River flow, as recorded at Vestal Station in the 1938-1967 period, were analyzed from the April through October period. River flow data were tabulated and plotted against rainfall for every rainy day for the 30-year period. Examination of these figures indicated that there was no correlation between rainfall and Susquehanna River flow. A possible physical explanation for this is that the storms during the April-October period are localized and do not cover a significant portion of the Susquehanna River drainage basin.

Another method for checking a correlation between rainfall and river flow was to test if the river flow ranges observed during rainy days were related in a systematic manner to the rainfall volume. This would indicate for instance, if higher river flows were observed more often during higher rainfall ranges. All of the river flows and rainfall volumes observed during rainy days in the 1938-1967 period were arranged as shown in Table VII-7. The most frequently observed river flow for each rainfall range is indicated by an asterisk. As rainfall volume increased, the most frequent river flow observed did *not* increase. This led to the conclusion that for higher volume storms in the Binghamton area, the river flow was not correspondingly in the higher range. In order words, rainfall volume and Susquehanna River flow in the Binghamton area were statistically independent. The pollution impacts of urban runoff would thus be more severe for rain storms of the local type that *do not substantially increase* river flows during critical low flow periods.

FREQUENCY OF OCCURRENCE OF LOCAL RAINSTORMS

The analysis of the number of occurrences of different rainfall intensities was essential in determining the design storm and for the appraisal of the impact of storm overflows on the river water quality.

Daily precipitation data were utilized as they are more representative for water quality calculations than hourly data. Storms might last for a few hours and more than one storm could occur in the same day. Furthermore, pollutant mass loadings from domestic or industrial sources are usually reported as daily averages.

TABLE VII-7

FREQUENCY OF RAINFALL VOLUMES AND RIVER FLOWS
NUMBER OF OCCURRENCES
(1938-1967)

Rainfall Range (Inch/Day)	Susquehanna River Flow Range (cfs) Below 500	500-700	700-1000	1000-2000	1500-2000	2000-3000	3,000-10,000	Total
0.5-0.8	13	18	27	28	24	16	33*	159
0.8-1.1	6	7	10	12	15	14	24*	88
1.1-1.4		1	6	10*		5	5	27
1.4-2.0	4	4	4	4	3	5	7*	31
Greater than 2.0		1	4	4*	2	1	1	13
							TOTAL	318

*Most probable river flow range for the rainfall range.

The months of April through September were considered in the analysis as this period covers summer recreational activities when high river quality is very desirable. Also, the lower river flows are usually in the end of the summer period making the effects of storm overflows on river water quality even more significant.

The length of the rainfall records analyzed was 22 years (1951-1973). Three precipitation levels were considered, 0.10", 0.50", and 1.00". The frequency of the number of days in one summer with precipitation equal to or exceeding a given level is shown in Table VII-8.

TABLE VII-8

MOST PROBABLE NUMBER OF DAYS IN A SUMMER PERIOD (APRIL-SEPTEMBER) WITH RAINFALL EXCEEDING DIFFERENT LEVELS

Daily Rainfall (inches)	No. of Days Rainfall is Exceeded
0.1	45
0.5	12
1.0	3

Rainfall intensity-duration-frequency data were compiled for the Binghamton area. Rainfall for durations from 30 minutes to 24 hours and return periods from 1 to 100 years are summarized in Table VII-9. The corresponding rainfall intensity is shown in Table VII-10.

SELECTION OF DESIGN CONDITIONS

Design conditions for any overflow management scheme include both storm and river flow characteristics. To evaluate the impact of a given storm on stream water quality and to determine the required effectiveness of a storm overflow control alternative, knowledge of the storm intensity, duration, river flow, initial dissolved oxygen concentrations, and temperature were needed.

TABLE VII-9

RAINFALL FOR DURATION FROM 30 MINUTES TO 24 HOURS AND RETURN PERIODS FROM 1 TO 100 YEARS IN THE BINGHAMTON AREA

Rainfall (Inch)

Return Period	30 Min.	1 Hr.	2 Hr.	3Hr.	6 Hr.	12 Hr.	24 Hr.
1 Yr.	0.75	0.90	1.2	1.35	1.75	2.0	2.4
2 Yr.	0.90	1.15	1.5	1.75	2.0	2.45	2.75
5 Yr.	1.20	1.55	1.85	2.20	2.50	3.0	3.60
10 Yr.	1.35	1.75	2.40	2.45	3.0	3.6	4.50
25 Yr.	1.50	2.05	2.50	2.95	3.5	4.1	5.0
50 Yr.	1.80	2.35	2.85	3.10	4.0	4.5	5.5
100 Yr.	2.05	2.50	3.45	3.5	4.5	5.2	6.2

Source: Reference IV-15. "Rainfall Frequency Atlas of the United States for durations from 30 Minutes to 24 hours and return periods from 1 to 100 years". Technical paper No. 40 U.S. Department of Commerce, Weather Bureau, May, 1961.

TABLE VII-10

RAINFALL INTENSITY-DURATION-FREQUENCY IN THE BINGHAMTON AREA

Rainfall Intensity (In/hour) for different durations

Return Period (Years)	30 Min.	1 Hr.	2 Hr.	3 Hr.	6 Hr.	12 Hr.	24 Hr.
1	1.50	0.90	0.60	0.45	0.23	0.17	0.10
2	1.80	1.15	0.75	0.58	0.33	0.20	0.11
5	2.40	1.55	0.92	0.73	0.42	0.25	0.15
10	2.70	1.75	1.30	0.82	0.50	0.30	0.19
25	3.20	2.05	1.35	0.98	0.60	0.34	0.21
50	3.60	2.35	1.42	1.03	0.67	0.38	0.23
100	4.10	2.50	1.72	1.17	0.75	0.43	0.26

Design River Flow

The occurrence of local rainstorms during periods of low river flows could result in low DO levels. Water quality standards are based on minimum average (seven consecutive days) river flow expected once in ten years (MA-7-CD-10). This flow, 330 cfs in the Susquehanna River at Vestal, was used in designing treatment levels for sewage treatment plants. However, as there are no State standards for the design of stormwater control facilities, and as storm overflows occur only periodically, the minimum average seven consecutive day flow occurring each year (MA-7-CD) was believed to be a more appropriate basis for design.

The daily rainfall record for the Binghamton area was examined to determine the rainfall occurrence during the yearly MA-7-CD flow. The maximum daily rainfall during that week was determined for each year for 35 years of records during the periods 1930-1952 and 1961-1972 as shown in Table VII-11. Flows in the Susquehanna River at Conklin during these MA-7-CD periods ranged from 100 to 700 cfs with 320 cfs being the average value. (Flow measurements were not available at Vestal for years after 1968).

The correlation between Susquehanna River flows at Vestal and those at Conklin indicate a ratio of Vestal to Conklin flow of approximately 1.9. Thus, the average of the MA-7-CD flows at Vestal would be approximately 600 cfs and was used as the design river flow.

The Susquehanna River temperature and initial DO levels used are 18 degrees C and 8.8 mg/l, respectively. These are the average characteristics in the Susquehanna River upstream of Binghamton for the summer months. The data were insufficient to determine these characteristics during the MA-7-CD flow. However, the DO for the MA-7-CD flow was expected to be similar to the average summertime flow as there are no significant wastewater discharges upstream of Binghamton.

Design Storm

If the treatment system is to be designed to achieve water quality levels in the Susquehanna River compatible with primary contact recreation (swimming), then the number of days when combined overflows are discharged directly to the river are of major importance and can be used in defining the "design storm."

TABLE VII-11

MAXIMUM DAILY RAINFALL DURING LOW FLOW PERIODS[1]

Year	**1930**	**1931**	**1932**	**1933**	**1934**	**1935**	**1936**	**1937**	**1938**	**1939**
Rainfall (inches)	0.20	0.14	0.38	1.61	0.46	0.21	0.04	0.11	0.78	0.42
Year	**1940**	**1941**	**1942**	**1943**	**1944**	**1945**	**1946**	**1947**	**1948**	**1949**
Rainfall (inches)	0.66	0.74	1.31	T	0.28	0.45	0.33	0.09	0.34	T
Year	**1950**	**1951**	**1952**	**1961**	**1962**	**1963**	**1964**	**1965**	**1966**	**1967**
Rainfall (inches)	0.87	0.62	0.01	0.12	0.26	–	0.44	0.91	0.50	0.16
Year	**1968**	**1969**	**1970**	**1971**	**1972**					
Rainfall (inches)	0.37	0.09	0.35	0.34	0.49					

[1] Low flow defined as minimum average seven consecutive day flow for each year.

Direct discharges would occur if the combined sewer overflow volumes exceeded the capacity of the overflow control measure. Previous studies indicate it is not economical to design the control measure to handle the most critical storm as most of the pollution load is discharged with the initial overflow flushes. Hence, to completely eliminate these direct discharges is not economically feasible or necessary. If a daily rainfall higher than one inch (Table VII-8) is used as the design storm, then the most probable number during the summer months for exceeding the control measure capacity would be three. Each exceedance results in raw sanitary discharge to the Susquehanna River rendering it unsafe for swimming. Since the impacts of these discharges could last for several days after each occurrence, the river might not be safe for swimming about ten days in the period of April to September. If a half inch of daily rainfall is considered as the design storm, then the most probable number of days exceeding overflow control measure capacities would increase to 12. This brings the number of days the river would not be safe for swimming to about 40 in the April to September period. A "design storm" of 1.25 inches per day (less than two occurrences during the summer months) was selected as being compatible for swimming and other body contact water recreation.

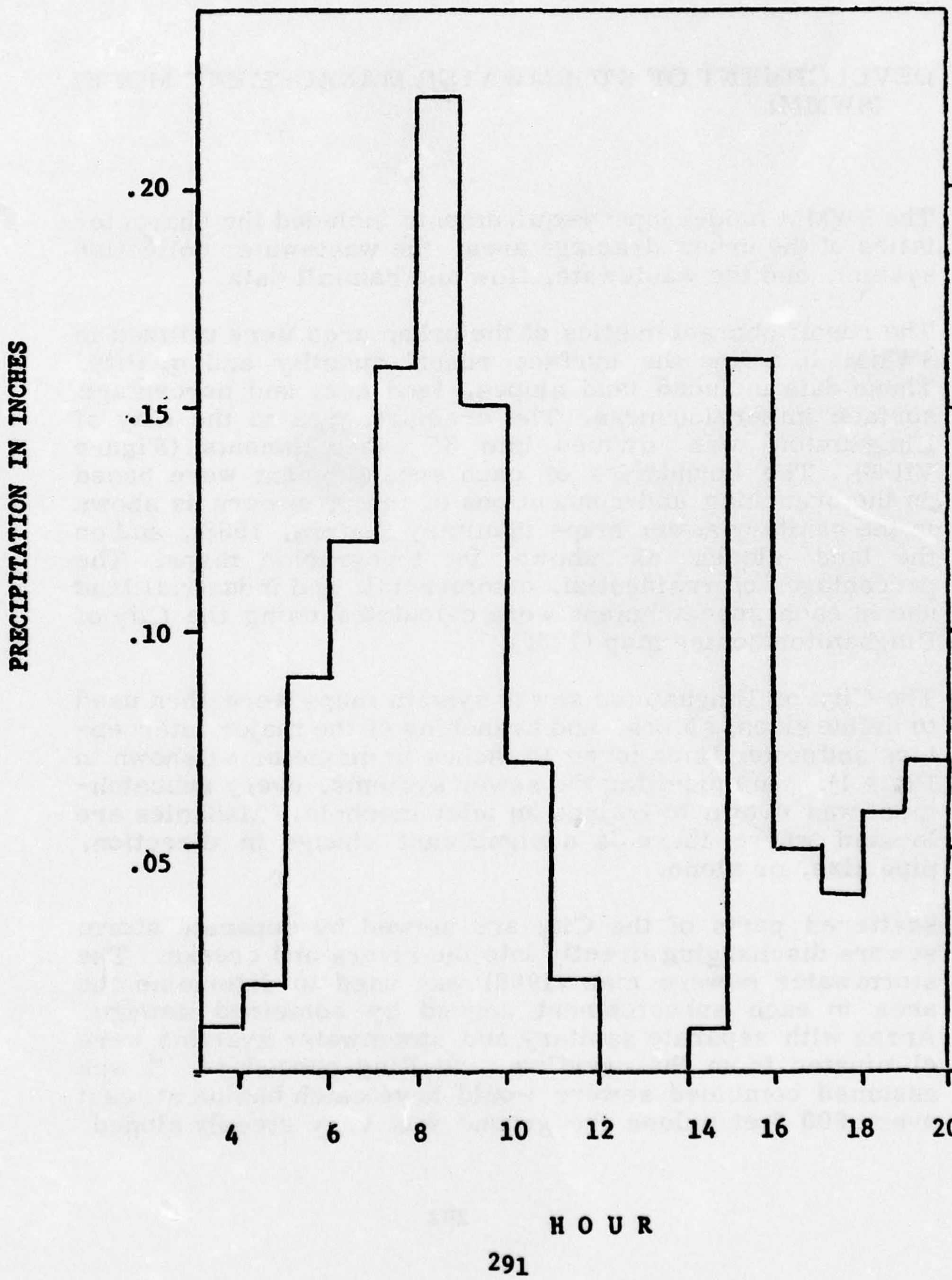
Overflow pollution would have its most harmful impact on river DO during low flow periods. The analysis of local storms (see Table VII-11) indicated rainfalls higher than 1.25 inches per day occurred only twice during the MA-7-CD flow period (the design river flow), for the 30 years of flow records analyzed. The probability of a 1.25 inch storm occurring during low flow periods, other than the yearly MA-7-CD flow, is, of course, greater. However, background river DO's also would be higher than during the MA-7-CD.

Analysis of coliform count, described in a previous section, and its correlation to storm occurrence indicated that a rainfall intensity of 0.10 in/hr may cause an order of magnitude increase in coliform count. Also, the analysis indicated that an increase of intensity above 0.15 in/hr will not significantly increase coliform count.

This leads to the conclusion that the "design storm" maximum intensity should be at least 0.15 in/hr. The design storm actually occurred on 29 October 1973 and its hyetograph is plotted in Figure VII-5. As shown in the figure, the maximum hourly intensity for this storm was 0.22 inches/hour. This intensity was used for design purposes.

FIGURE VII -5

HOURLY PRECIPITATION FOR THE
DESIGN STORM



In summary, the design conditions for the stormwater management analysis were:

River Flow: 600 cfs at Vestal

Storm: 1.25 inches/24 hours and maximum hourly intensity of 0.22 inches.

DEVELOPMENT OF STORMWATER MANAGEMENT MODEL (SWMM)

The SWMM model input requirements included the characteristics of the urban drainage area, the wastewater collection system, and the wastewater flow and rainfall data.

The runoff characteristics of the urban area were utilized in SWMM to define the surface runoff quantity and quality. These data included land slopes, land use, and percentage surface imperviousness. The drainage area in the City of Binghamton was divided into 35 subcatchments (Figure VII-6). The boundaries of each subcatchment were based on the branching and connections of major sewers as shown in the sanitary sewer maps (Sanitary Sewers, 1968), and on the land slopes as shown in topographic maps. The percentages of residential, commercial, and industrial land use in each subcatchment were calculated using the City of Binghamton zoning map (1969).

The City of Binghamton sewer system maps were then used to define sizes, slopes, and branching of the major interceptors and sewer lines (over 15 inches in diameter as shown in Plate 1). In modelling the sewer systems, every subcatchment was drawn to include an inlet manhole. Manholes are located where there is a significant change in direction, pipe size, or slope.

Scattered parts of the City are served by separate storm sewers discharging directly into the rivers and creeks. The stormwater sewers map (1968) was used to determine the area in each subcatchment served by combined sewers. Areas with separate sanitary and stormwater systems were eliminated from the overflow modelling procedure. It was assumed combined sewers would have catch basins at least every 600 feet unless the ground was very steeply sloped.

CITY OF BINGHAMTON

STORM WATER SYSTEM AS MODELLED FOR SWMM

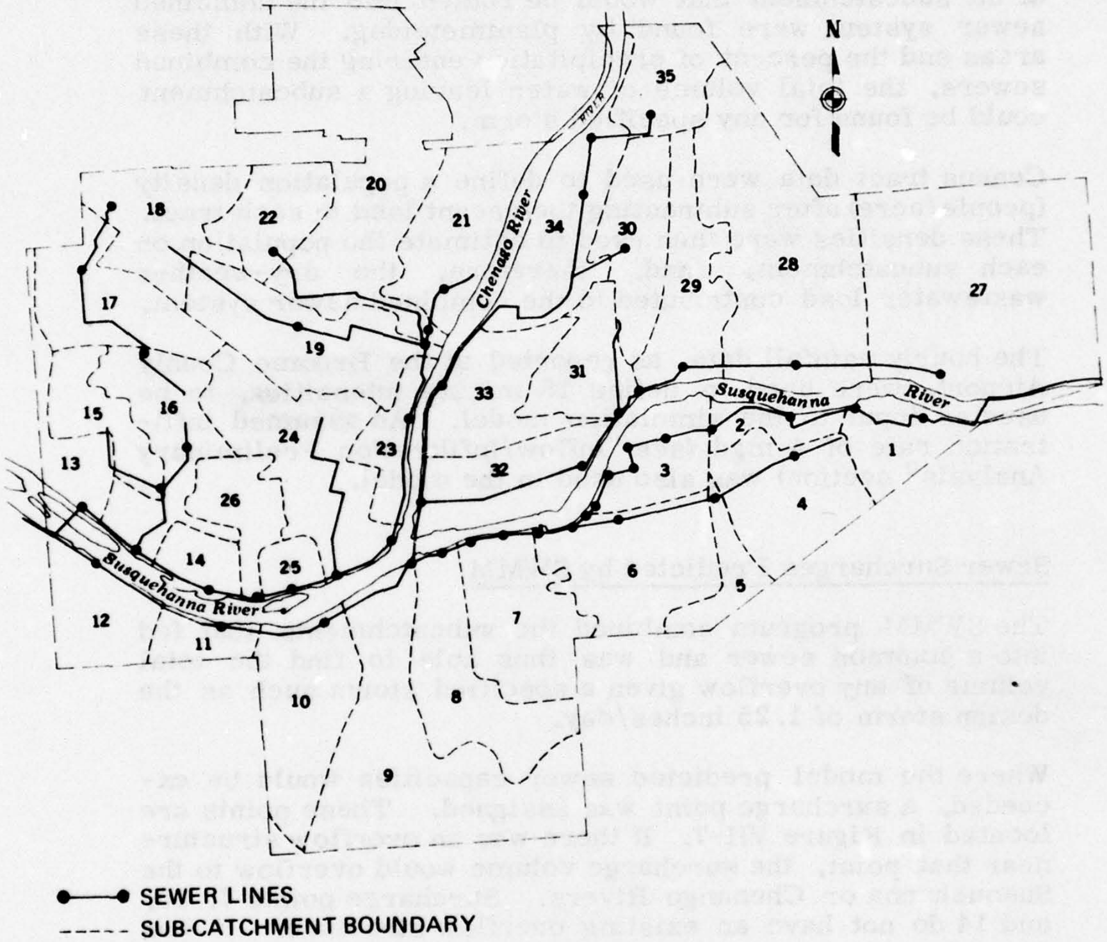


Figure VII-6

The impervious ratios of the subcatchment areas had to be found so that the quantity of precipitation leaving the subcatchment in a combined sewer could be calculated. The land uses and housing densities were determined for each subcatchment area using photographic maps (1973) and census tract data. These land uses were assigned estimated imperviousness ratios which were used to produce an average impervious value for each subcatchment as shown in Table VII-12. With these ratios, the SWMM computed the runoff from the pervious and impervious areas (after subtracting detention losses) for each subcatchment. The areas of the subcatchment that would be routed into the combined sewer system were found by planimetering. With these areas and the percent of precipitation entering the combined sewers, the total volume of water leaving a subcatchment could be found for any specified storm.

Census tract data were used to define a population density (people/acre) after subtracting the vacant land in each tract. These densities were then used to estimate the population on each subcatchment, and, therefore, the dry-weather wastewater load contributed to the combined sewer system.

The hourly rainfall data, as reported at the Broome County Airport, were used to define 15 minute intensities, to be used as input to the simulation model. An assumed infiltration rate of 4 mgd (see "Inflow/Infiltration Preliminary Analysis" section) was also used in the model.

Sewer Surcharges Predicted by SWMM

The SWMM program combined the subcatchments that fed into a common sewer and was thus able to find the total volume of any overflow given a specified storm such as the design storm of 1.25 inches/day.

Where the model predicted sewer capacities would be exceeded, a surcharge point was assigned. These points are located in Figure VII-7. If there was an overflow structure near that point, the surcharge volume would overflow to the Susquehanna or Chenango Rivers. Surcharge points 7, 10, and 14 do not have an existing overflow structure. It was found that the sewer capacities at these locations are indeed exceeded during storms, causing local flooding conditions.

TABLE VII-12

ESTIMATES OF THE IMPERVIOUSNESS RATIOS FOR THE CITY OF BINGHAMTON SUBCATCHMENTS

	PERCENTAGE LAND USE PER SUBCATCHMENT AND IMPERVIOUS RATIO FOR EACH			
Subcatchment Section	**Land Use* Category**	**Percentage**	**Impervious Ratio for Land Use**	**Average Impervious Ratio for Subcatchment**
1	O.S.	44	0	
	I	46	0.7	0.35
	R	10	0.3	
2	O.S.	100	0	0
3	O.S.	25	0	
	I	50	0.7	0.43
	R_3	25	0.3	
4	O.S.	10	0	
	R_3	60	0.3	0.30
	R_H	30	0.4	
5	$R_{1.6}$	100	0.25	0.25
6	O.S.	40	0	
	R_2	60	0.27	0.16
7	$R_{(4.5)}$	100	0.34	0.34
8	$R_{(4.5)}$	100	0.34	0.34
9	O.S.	25	0	0.22
	$R_{(3.2)}$	75	0.3	
10	O.S.	75		
	$R_{(2.2)}$	25	0.28	0.07
11	$R_{(0.5-1)}$	100	0.2	0.20
12	$R_{(1)}$	100	0.22	0.22
13	O.S.	25	0	
	$R_{(2-3)}$	75	0.28	0.21
14	O.S.	40	0	
	I	20	0.7	0.24
	$R_{(1-2)}$	40	0.24	
15	$R_{(3-4)}$	100	0.32	0.32
16	$R_{(3-4)}$	100	0.32	0.32
17	I	10	0.7	
	$R_{(3-4)}$	90	0.32	0.36
18	I	100	0.7	0.7
19	I	50	0.7	
	$R_{(4.5)}$	50	0.33	0.51

TABLE VII-12 (cont)

ESTIMATES OF THE IMPERVIOUSNESS RATIOS FOR THE CITY OF BINGHAMTON SUBCATCHMENTS

Subcatchment Section	PERCENTAGE LAND USE PER SUBCATCHMENT AND IMPERVIOUS RATIO FOR EACH			Average Impervious Ratio for Subcatchment
	Land Use* Category	Percentage	Impervious Ratio for Land Use	
20	O.S. I $R_{(4)}$	50 20 30	0 0.7 0.32	0.24
21	O.S. $R_{(2)}$	80 20	0 0.27	0.05
22	$R_{(4-5)}$	100	0.33	0.33
23	I $R_{(H)}$	90 10	0.7 0.40	0.67
24	O.S. $R_{(3-4)}$	5 95	0 0.32	0.30
25	$R_{(2-3)}$	100	0.28	0.28
26	$R_{(3-4)}$	100	0.32	0.32
27	O.S. R	50 50	0 0.22	0.11
28	I $R_{(2.6)}$	10 90	0.7 0.2	0.25
29	I $R_{(5)}$	80 20	0.7 0.34	0.63
30	I	100	0.70	0.70
31	I $R_{(3.6)}$	30 70	0.7 0.32	0.43
32	I	100	0.70	0.70
33	I $R_{(6)}$	75 25	0.70 0.35	0.61

TABLE VII-12 (cont)

ESTIMATES OF THE IMPERVIOUSNESS RATIOS FOR THE CITY OF BINGHAMTON SUBCATCHMENTS

	PERCENTAGE LAND USE PER SUBCATCHMENT AND IMPERVIOUS RATIO FOR EACH			
Subcatchment Section	Land Use* Category	Percentage	Impervious Ratio for Land Use	Average Impervious Ratio for Subcatchment
34	O.S.	15	0	
	I	10	0.7	0.31
	$R_{(4)}$	75	0.32	
35	O.S.	45	0	
	I	25	0.7	0.29
	$R_{(3.3)}$	15	0.31	
	$R_{(H)}$	15	0.41	

* O.S. = Open Space
I = Industrial or Commercial Land Use
R(n) = Residential with density of n houses/acre
R(H) = Residential high density high rise buildings

CITY OF BINGHAMTON
COMBINED SEWER SURCHARGE POINTS
(AS PREDICTED BY SWMM RESULTING FROM 1.25 INCH STORM)

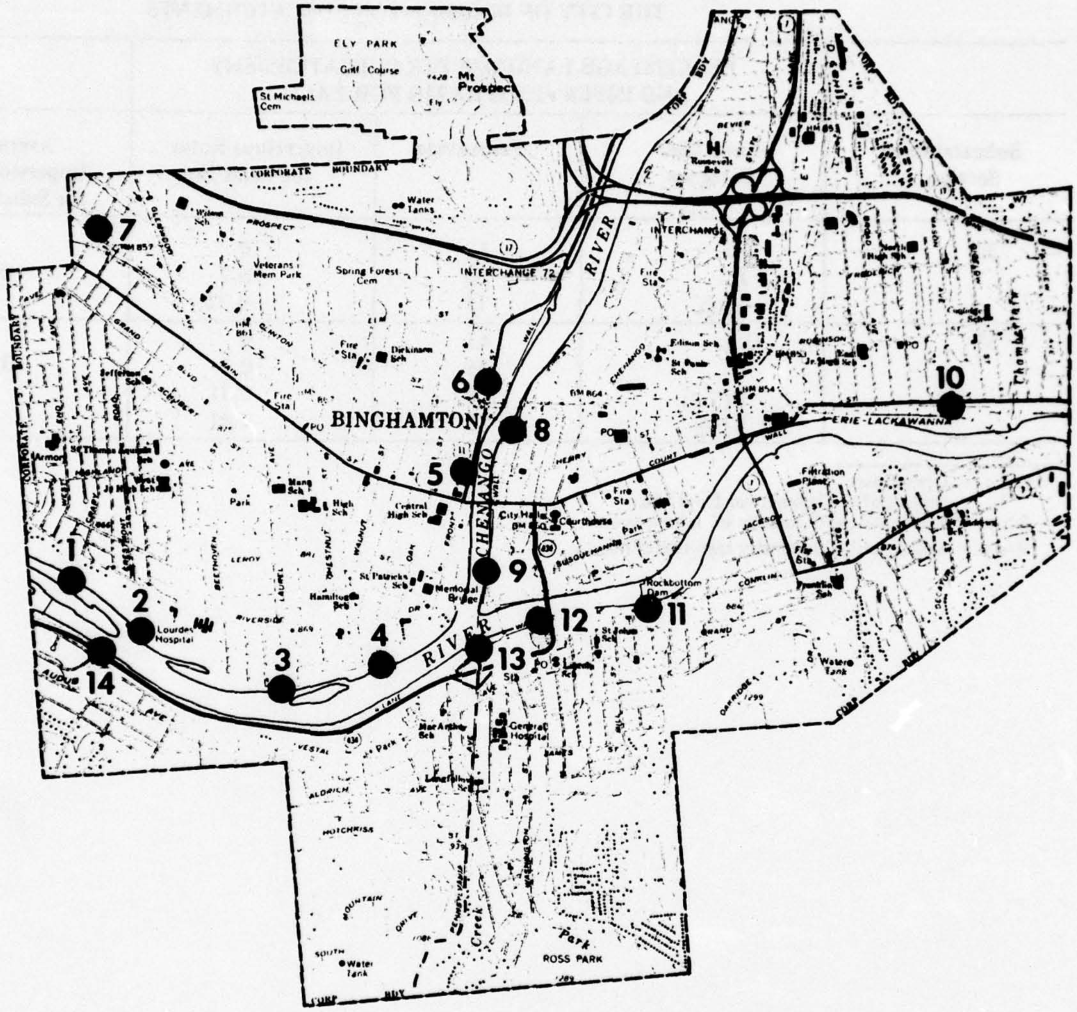


FIGURE VII-7

Average Overflow Volume and Pollutants Loads Per Season in the Binghamton Area

The overflow volume and BOD resulting from various storms were estimated using the Storm Water Management Model and are shown in Table VII-13.

Estimates of BOD quantities were based on the ratio of storm runoff to domestic wastewater and the average strength of the domestic flow. These estimates, combined with the most probable number of occurrences of the corresponding storm, were used to define the annual volume and loads summarized in Table VII-14. The volume and BOD mass of the overflows (from Table VII-13) are plotted as functions of rainfall in Figures VII-8 and VII-9, respectively.

The Binghamton-Johnson City wastewater treatment plant maintains a continuous flow record. The storm of October 29, 1973, resulted in a rainfall of 1.25 inches and the recorded treatment plant flow was compared to the independent SWMM flow predictions. The correlation between flows was excellent (Figure VII-10).

DYNAMIC WATER QUALITY MODEL--RECEIVE II

The quantity analysis of the Receive II model assumed the river as a series of prismatic rectangles. These prismatic rectangles were the channels, and their junctions were the nodes. To set up the program, specific data on the channels and nodes were required. The channel length, width, average depth, and roughness coefficient (Manning 'n') must be determined.

In the Susquehanna River, an attempt was made to have nodes and, therefore, cross sections, wherever there would be a significant input into the river. Cross sections that were repetitive, or not characteristic were eliminated. Each cross section was evaluated under the design flow for storm conditions, i.e., 600 cfs (at Vestal). The energy slope and Manning 'n' were taken from a flood plain study done by the Baltimore District, US Army Corps of Engineers. The model assumed the depth would approximate the hydraulic radius. The depth and width of a representative rectangle was the one that fit the equation with the least change in the shape and area of the channel.

TABLE VII-13

CITY OF BINGHAMTON
COMBINED SEWER OVERFLOW VOLUME, BOD AND NOD ASSOCIATED WITH STORMS OF DIFFERENT INTENSITIES

Rainfall (inches/ 24 hours)	Runoff Volume (Million Gallons)	Overflow Volume (Million Gallons)	Overflow BOD (lb)	Increased STP BOD (lb)	Total BOD (lb)	Overflow NOD (lb)	Increase in STP NOD (lb)	Total NOD (lb)
0.3	5	3	4,000[1]	400	4,400	2,640	50	2,690
0.7	16	8	7,000[2]	1,200	8,200	4,620	100	4,720
1.4	34	17	7,200[2]	3,000*	10,200	4,700	200	4,900

[1] Assume overflow BOD = 150 mg/l
[2] Assume overflow BOD = 100 mg/l

TABLE VII-14

CITY OF BINGHAMTON, AVERAGE SEASONAL OVERFLOW VOLUME, BOD AND NOD

Rainfall (inches/ 24 hours)	Most Probable No. of days per season (April-October)	Overflow Volume (Million Gallons)	BOD (10^3 lbs)	NOD (10^3 lbs)
0.3	33	99	145	89
0.7	9	72	74	43
1.4	3	51	31	15
Total		222	250	147

RELATIONSHIP BETWEEN STORM OVERFLOW VOLUME AND RAINFALL (SWMM MODEL PREDICTIONS)

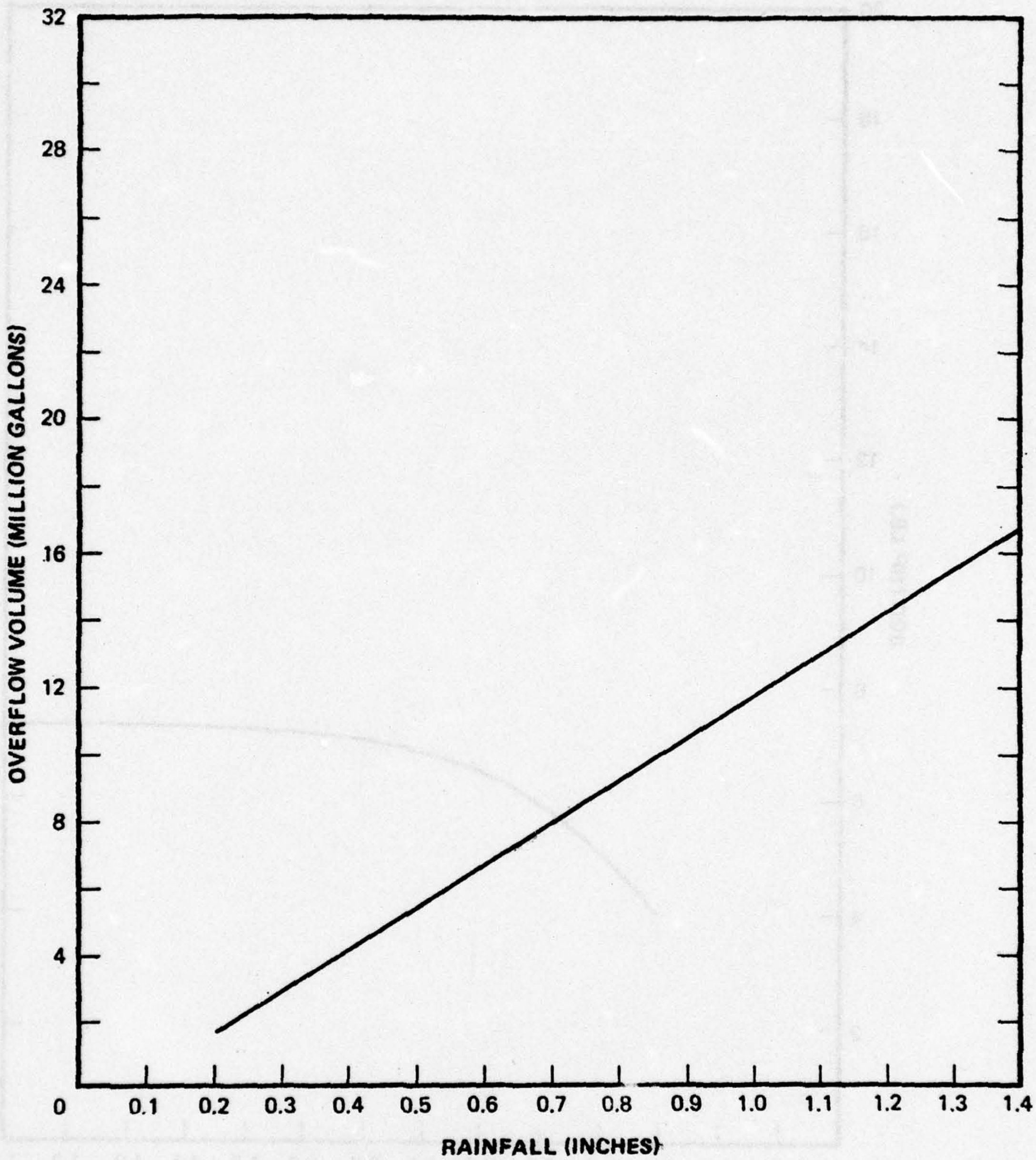


FIGURE VII-8

RELATIONSHIP BETWEEN STORM OVERFLOW BOD MASS AND RAINFALL (SWMM MODEL PREDICTIONS)

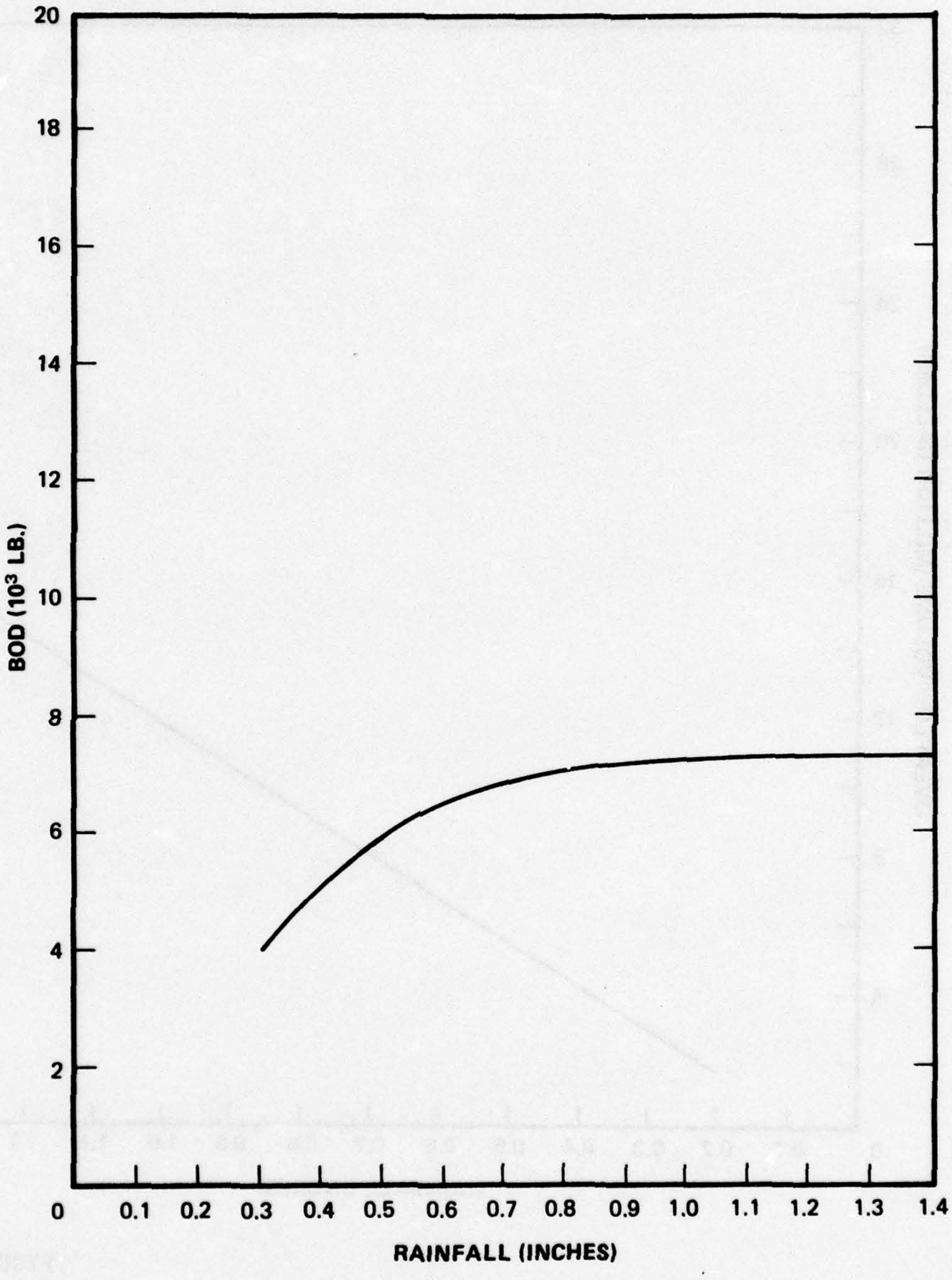


FIGURE VII-9

VERIFICATION OF BINGHAMTON–JOHNSON CITY WASTEWATER TREATMENT PLANT FLOW AS PREDICTIED BY SWMM (1.25 INCH STORM OF OCTOBER 29, 1973)

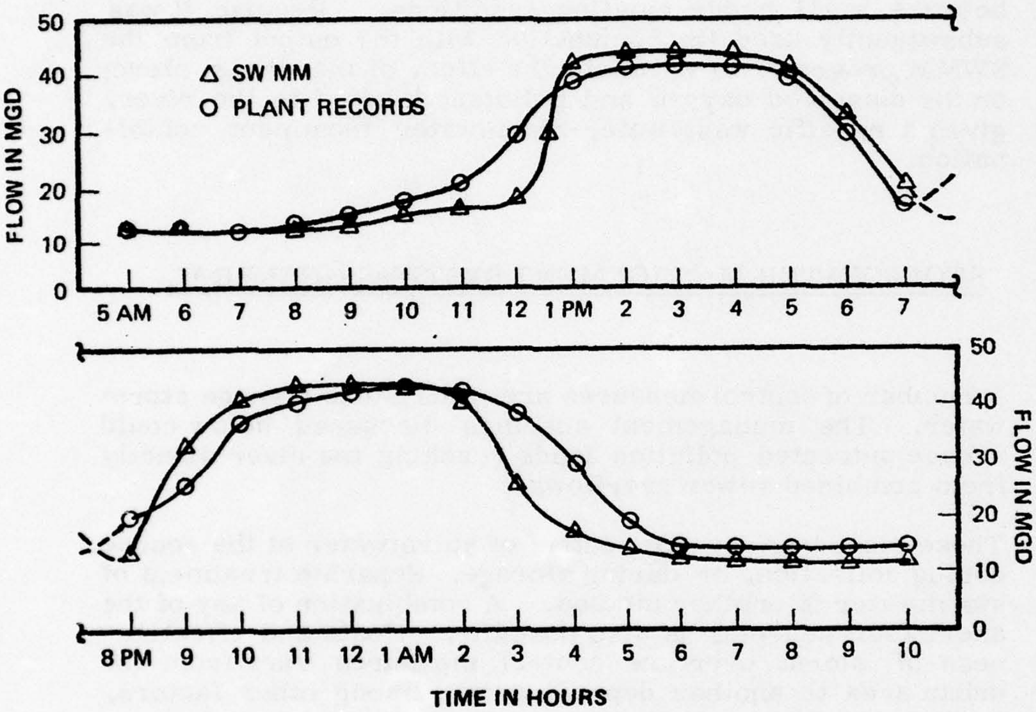


FIGURE VII-10

An analysis of storms expected to occur during periods of low river flow and the resulting loading of BOD and NOD to the river was computed by SWMM. This pollutant load was then added to the Binghamton-Johnson City treatment plant which is attaining 90 percent removal of BOD and NOD during summer dry weather conditions. Thus, the amount of oxygen demanding effluent discharge is about 10 percent of the raw waste load. This treatment plant effluent discharge is expected to deplete the river DO to 4-5 mg/l during MA-7-CD-10 flows under existing conditions, and 3-4 mg/l by the year 2020. Overflow loads up to four times the load of treatment plant effluent could be expected during storms coinciding with the MA-7-CD-10 flow, resulting in DO levels below 4 mg/l under existing conditions. Receive II was subsequently used (in conjunction with the output from the SWMM program) to estimate the effect of the design storm on the dissolved oxygen and pollutant loading to the river, given a specific wastewater-stormwater treatment combination.

STORMWATER MANAGEMENT SYSTEMS--GENERAL

A number of control measures are available to manage storm water. The management schemes discussed below could reduce untreated pollution loads reaching the river directly from combined sewer overflows.

These measures involve control of stormwater at the source, during collection, or during storage. Separate treatment of stormwater is another method. A combination of any of the above four schemes is also possible. Costs and effectiveness of storm overflow control measures vary from one urban area to another depending on, among other factors, land availability, construction costs, topography, and storm characteristics.

SOURCE CONTROLS

Source controls limit the quantity and improve the quality of storm runoff before it enters the collection system. Examples of these measures include surface detention for runoff control, erosion control, restrictions on chemical use and improved neighborhood sanitation practices.

COLLECTION SYSTEM CONTROLS

Examples of collection systems controls include sewer system separation, inflow/infiltration control, improved regulation devices, temporary increased line-carrying capacities using polymer additives, in-line storage, flushing and improved sewer cleaning practices, and use of remote monitoring/control systems. The emphasis is upon utilization of the existing facilities for optimal conveyance and possible in-line storage of flows.

System control utilizing in-line storage represents promising alternatives in combined or partially combined sewer systems. The intent is to assist in routing and storing stormwater flows to make the most effective use of interceptor capacities. The necessary components include both sensing (e.g., flow levels, rates, quality, gate positions, and rainfall), and control devices (gates, valves, inflatable dams, regulators, and pumps) operable from a central facility. The concept of constructing new separate sanitary sewers to replace existing conbined sewers largely has been abandoned because of the enormous cost, inconvenience to the public, extended time required for implementation, and because of the pollution potential of untreated urban storm runoff on receiving waters.

STORAGE

Storage facilities possess many of the favorable attributes desired in stormwater treatment. They are capable of providing flow equalization and attenuation, and, in the case of tunnels, flow transmission; they are simple to design and operate; they respond without difficulty to intermittent and random storm behavior; they are relatively unaffected by flow and quality changes; and frequently, they can be operated in concert with regional dry weather flow treatment plants for benefits during both dry and wet weather conditions. Storage basin facilities for storm overflow control may be used during dry weather flow conditions in equalizing the diurnal variations observed in domestic wastewater flows. This will reduce the peak flows at the domestic wastewater treatment plant, and thus may delay expansion of facilities due to projected wastewater flow increases. Also equalizing the flow rates may improve the efficiency of the wastewater treatment processes. Disadvantages of storage facilities include their large size, high cost, and

dependency on other treatment facilities for dewatering and solids disposal. Storage facilities include concrete holding tanks, open basins, tunnels, underground and underwater containers, abandoned facilities, and existing sewer lines.

TREATMENT

Physical Treatment

Physical treatment processes in many ways are well suited to stormwater applications, particularly with respect to solids removal. These processes include sedimentation, dissolved air flotation, screening, filtration, and special regulator devices.

Biological Treatment

Examples of biological treatment applications to stormwater include the contact stabilization modification of activated sludge, high rate trickling filtration, bioadsorption using rotating biological contactors, and oxidation lagoons of various types. The first three are operated conjunctively with dry weather flow plants to supply the biomass and the fourth involves long term storage.

Physical/Chemical Treatment

Physical/chemical processes are of *particular importance* to stormwater treatment because of their adaptability to automated operation, rapid start-up and shutdown characteristics, and very good resistance to shock-loads. The most promising combination includes chemical clarification, filtration, and adsorption of activated carbon. Use of activated carbon columns, during non-storm periods, for polishing secondary effluent achieves a dual advantage.

Disinfection

The most commonly used disinfectants are ozone and chlorine compounds: sodium, or calcium hypochlorite, and chlorine dioxide. Because of safety in storage and transport, the use of hypochlorites predominates in current full scale applications.

Integrated Systems

Some of the facilities constructed for the control of storm overflow of the combined sewer system can be effectively utilized in an integrated system to manage both dry weather and wet weather wastewaters.

A combination of control and treatment systems through dual use of storm control facilities is possible during dry weather flow for upgrading treatment performance. These systems employ some form of storage and require mathematical models for development and operation. These integrated approaches are being demonstrated in several urban areas.

Interceptors included in storm overflow control measures could replace some of the existing trunk and interceptor lines that are old and subject to high infiltration rates. This may prove effective in controlling significant amounts of groundwater infiltrating into sewer lines.

COSTS AND EFFECTIVENESS OF TYPICAL CONTROL MEASURES

The costs and effectiveness will vary from one urban area to another depending on, among other factors, land availability, construction cost, topography, and storm characteristics. These data are mostly from demonstration projects; however, the number of projects is not enough to cover the whole range of variations expected in different site applications.

Storage

Storage may be in-system or off-system. Detroit found the cost of in-system controlled storage to be as low as $0.02 to $0.04 per gallon of storage based upon the most favorable sites. This range was approximately 1/10 the estimated cost of large off-line facilities. Representative storage costs from other projects are summarized in Table VII-15. Storage also provided some removal of sediment.

TABLE VII-15

STORAGE COSTS FOR VARIOUS CITIES[1]

Location	Storage Million Gallons	Capital Cost $[2]	Storage Cost[2] $/gal.
Seattle, Washington			
Control and monitoring system	–	3,728,000	–
Automated regulator station	–	4,154,000	–
	32.0	7,882,000	0.25
Minneapolis-St. Paul, Minn.	–	3,195,000	–
Chippewa Falls, Wisconsin Storage	2.8	792,000	0.28
Jamaica Bay, N.Y., N.Y.			
Basin	10.0	22,578,000	2.26
Basin and sewer	23.0	22,578,000	0.98
Humboldt Avenue, Milwaukee, Wisconsin	4.0	2,141,000	0.53
Boston, Massachusetts			
Cottage Farm Stormwater Treatment Station[3]	1.3	6,603,000	5.05
Chicago, Illinois			
Storage basins	2.74	605,000	0.22
Collecting, tunnel, and pumping[4]	2.83	805,000	0.29
	5.57	1,410,000	0.25

[1] Source: J.A. Lager and R. Field, "Counter Measures for Pollution from Overflows," Journal Water Pollution Control Federation, October, 1973.
[2] ENR = 2135
[3] Includes pumping station, chlorination facilities, and outfall.
[4] Includes 193.1 km (120 miles) of tunnels.

Treatment

The hypothetical comparison of alternative treatment processes based on an assumed 25 mgd installation is summarized in Table VII-16. Since the feed to the units varies continuously in both quantity and quality, efficiencies should be regarded as gross indicators only. Similarly, the cost estimates are based on one or at most only a few installations, and may not be fully representative in terms of completeness of facilities and difficulties encountered in construction. Preliminary selection of the alternative treatment processes should be based on the pollutant removal efficiencies of each process and the cost per unit mass of pollutant removed.

Some of the treatment processes summarized in Table VII-16 are not efficient for the purpose of the Study. For example, the percentage removal of BOD is too low for the ultra-fine screen treatment process to be an effective control measure. For the ultra-high rate filter, the cost per million gallons per day per one percent BOD removal is higher than any other treatment process while the BOD and SS removal efficiency is lower than many of these processes. Also, the high rate trickling filter does not compare favorably with the contact stabilization process as it has lower removal efficiency and is more costly per unit BOD removal.

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM (NPDES) PERMITS

According to the requirements of the NPDES special permits for combined sewer overflows and/or bypasses, overflow management programs should "...eliminate or significantly reduce pollution from these sources so as to maximize the achievement of water quality standards." It should be noted here that the emphasis is to achieve water quality standards. Secondly, it is recognized that elimination of overflow pollution may prove to be uneconomical and, hence, significant reduction in the load may be more practical than complete elimination.

The NPDES permits also require a final plan describing the overflow management program. This plan should include the evaluation of the following measures:

TABLE VII-16

TREATMENT PROCESSES COSTS AND EFFECTIVENESS[1]

Process	Estimated Removal % BOD	Suspended Solids	Capital Cost per mgd[2]	Capital Cost per mgd per 1% BOD Removal
Dissolved Air Flotation[3]	40	60	$37,300	933
Same w/Chemical addition	52	78		
Micro-strainer	10-50	70	13,900	463
Ultrafine Screen	15	40	8,300	553
Ultrahigh Rate Filter	8-36	38-73	67,200[4]	2,864
Chemical Clarification	60	60	57,600	960
Contact Stabilization	83	92	83,500	1,006
High Rate Trickling Filter	65	65	72,600	1,117

[1] Source: Lager and Field, 1973.
[2] Based on 25 mgd facility, ENR = 2135.
[3] Includes ultrafine screens as pretreatment.
[4] Extrapolated from bench scale data.

1. Dual use treatment facilities.

2. Storing and/or treating initial or final sewer system flushes.

3. Storage and subsequent treatment of discharges.

4. Improvements in the sewer system.

OVERFLOW CONTROL ALTERNATIVES FOR THE CITY OF BINGHAMTON

The control measures considered included both storage and treatment. The required capacities associated with these control measures were based on the overflow volumes and rates as estimated by the Storm Water Management Model. Cost data for these storage and treatment facilities and the effectiveness of these alternatives in controlling overflows is discussed. An alternative to either storage or treatment, sewer system separation, was not considered in detail because of the prohibitive high cost, inconvenience to the public, and the uncertainty of the quality of urban storm runoff to allow its direct discharge to streams.

The effectiveness of a combined overflow control alternative was measured by both the percentage reduction in the long-term overflow pollution load (level of treatment) and by its ability to prevent direct discharges of overflows resulting from larger storms (system capacity). The latter measure is related to the selection of the "design storm." The alternatives discussed below were all based on the same "design storm" defined as 1.25 inches of rainfall in 24 hours. The overflow control alternatives were sized not only to handle combined sewer overflows but also the increased flow to the Binghamton-Johnson City Sewage Treatment Plant as a result of the design storm. This would reduce the need for the treatment plant to be bypassed (with untreated flows discharged directly to the river), particularly when future service area expansion results in increased flows.

PROPOSED ALTERNATIVES

Four alternative control measures for the City of Binghamton's combined sewer overflow problems were developed and evaluated. The control structures were designed to handle the overflow volume and the amount of increased flow occurring at the treatment plant as a result of the design storm.

The four stormwater management alternatives were:

1. Storage and subsequent treatment at Binghamton-Johnson City Sewage Treatment Plant.

2. Treatment at overflow sites using micro-strainers.

3. Treatment at overflow sites using dissolved air flotation.

4. Centralized treatment using a modified biological process.

Chlorination facilities were included in all the alternatives for disinfection.

In the following sections, the description of the location, the capacity and the main characteristics of the facilities required for each storm overflow management scheme are summarized and the effectiveness of each alternative is discussed. One measure of system effectiveness was its ability to achieve dissolved oxygen levels compatible with those required by the water quality standards. Another measure was the ability to reduce the pollution loads to the Susquehanna River. The load reduction should be evaluated in the short term as well as in the long term. The short term evaluation reflects the reduction of the shock load due to the storm event, i.e., immediately following a given storm. The long term evaluation reflects the total load reduction due to the typical storms that may occur in the April through September period.

Finally, cost estimates of each alternative are summarized. This includes the capital, operation and maintenance, and annual costs. Also, a unit cost analysis was performed in order to compare the cost-effectiveness of the storm overflow control alternatives to those of the dry weather wastewater management alternatives.

Alternative A--Storage and Subsequent Treatment at Existing Treatment Plant

General.

Storage basins may be built near the locations where combined sewer overflows are discharged or near major sewers that carry substantial flows during wet weather conditions. During these high sewer flow conditions, overflows would be diverted to the basins. After the storm subsides, the stored wastewater would be released to the existing Binghamton--Johnson City sewage treatment plant during periods of low sanitary flow. This would reduce the peak flows at the plant, and thus might delay required expansion of facilities due to projected wastewater flow increases. Also, equalizing the flow rates could improve the efficiency of the wastewater treatment processes.

It is advantageous to have the storage basins as close as possible to the main interceptors in order to save the transmission costs to and from these basins. The exact location of the overflow storage basins would be influenced by land availability. It might prove infeasible to build a storage basin at a specific overflow site. In this case, it could be possible to build the basin further upstream in the sewer system or build a transmission line from the overflow point to the available storage site.

The cost of building a storage facility, of a given size, is highly dependent on local conditions such as land availability and costs, groundwater conditions, soil, type of storage facility, and local construction costs. It is possible to build a storage facility under or above the ground surface and the choice between these two types depends on the economic and environmental impacts of each. In the cost analysis below, the cost was based upon that of a covered concrete storage facility built with its top at ground level.

The effectiveness of the storage alternative in reducing the combined overflow pollution loads was determined by the removal efficiencies of the existing Binghamton-Johnson City treatment facility. Thus, this alternative could achieve combined overflow effluent quality characteristics similar to those of secondary effluent.

Main Characteristics.

In this scheme, combined overflows would be stored in five storage basins located as shown in Figure VII-19, near the

overflow sites. Each storage basin was designed to contain the stormwater runoff volume from the design storm of 1.25 inches in a 24 hour period. These volumes were defined using a modified version of the SWMM. A flow diagram for the storage basin facility is shown in Figure VII-11.

Bar screens would be used to provide preliminary treatment of the overflows before storage. Aeration facilities would be provided to prevent odor problems from the stored overflows. Removal facilities would clean the solids deposited after the basins are dewatered. Chlorination facilities at the overflow bypass would be used only if storm overflow volume exceeds the basin capacity.

The system operates such that the combined sewer flow would be directed to the storage basin following the storm occurrence. The stored volume would be released from the basin in the following days during dry weather flows (municipal wastewater, low flow period, utilizing the diurnal variation in sanitary flow).

Effectiveness.

The minimum instantaneous DO level during the design storm (1.25 inch/day) would be between 5 mg/l and 6 mg/l in conjunction with any of the dry weather wastewater treatment alternatives.

Reduction of pollution loads due to combined sewer overflows are as follows:

a. Short Term (reductions per storm):

BOD Load Reduction = 10,160 lbs.

NOD Load Reduction = 6,200 lbs.

Coliform--Reduce effluent total count to 1,000/100 ml (at STP).

b. Long Term: This alternative would reduce the BOD pollution discharged to the Susquehanna River by an average of 225,000 pounds during a typical summer (April-September) period. This reduction would be equivalent to 42 percent of the current 3,000 lb/day daily effluent BOD load from the Binghamton-Johnson City STP. It would also reduce the NOD load discharged to the Susquehanna River by an average of 75,000 pounds during a typical summer period, which is equivalent to 8 percent of the current 5,200 lb/day discharged in the secondary effluent of the STP.

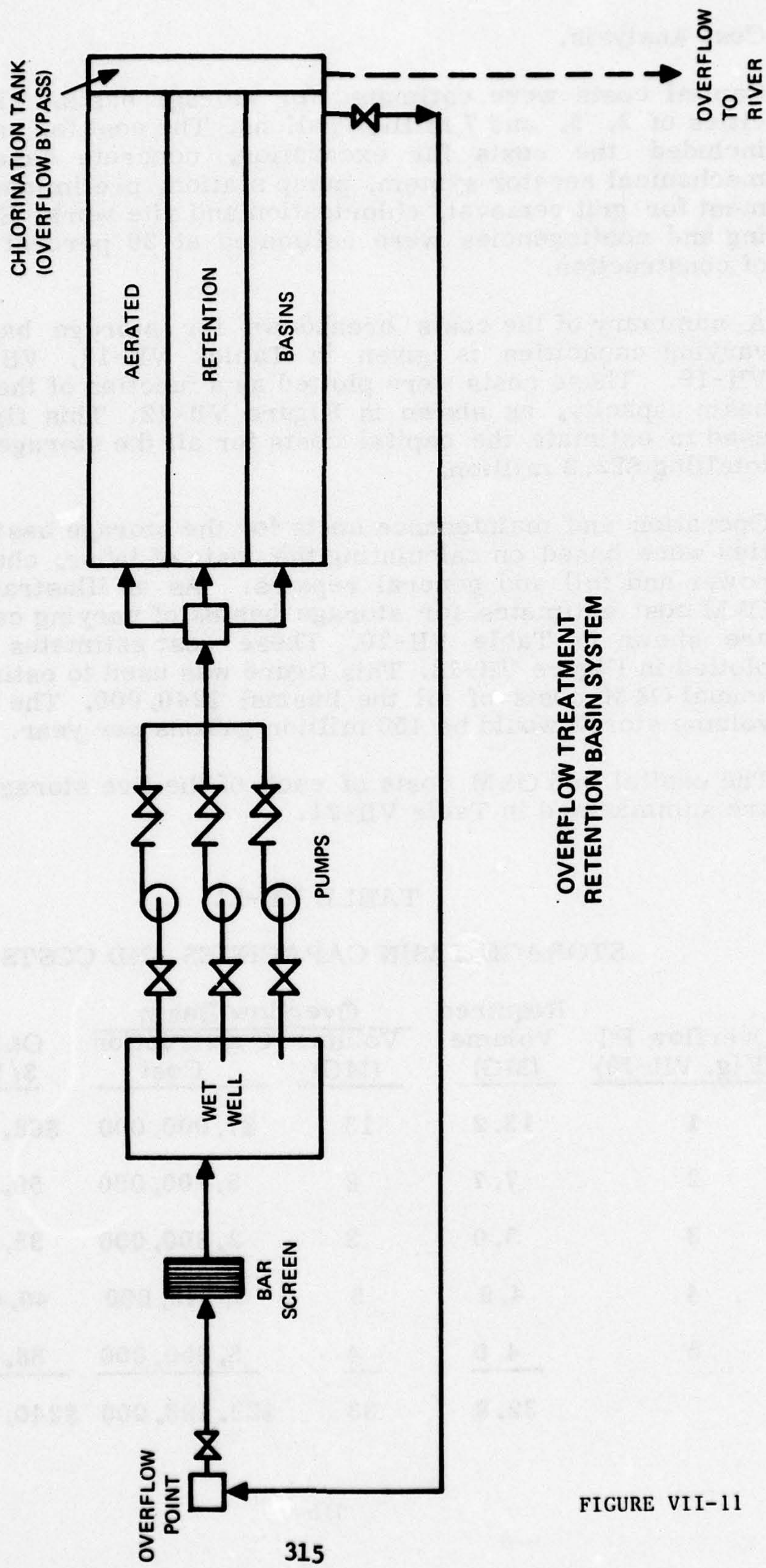


FIGURE VII-11

Cost Analysis.

Capital costs were estimated for storage basins with capacities of 2, 5, and 7 million gallons. The cost for each basin included the costs for excavation, concrete base piling, mechanical aerator system, pump station, preliminary treatment for grit removal, chlorination and site work. Engineering and contingencies were estimated at 30 percent of costs of construction.

A summary of the costs breakdown for storage basin with varying capacities is given in Tables VII-17, VII-18, and VII-19. These costs were plotted as a function of the storage basin capacity, as shown in Figure VII-12. This figure was used to estimate the capital costs for all the storage basins, totalling $22.2 million.

Operation and maintenance costs for the storage basin facilities were based on calculating the costs of labor, chemicals, power and full and general repairs. As an illustration, the O&M cost estimates for storage basins of varying capacities are shown in Table VII-20. These cost estimates are also plotted in Figure VII-13. This figure was used to estimate the annual O&M costs of all the basins: $240,000. The average volume stored would be 450 million gallons per year.

The capital and O&M costs of each of the five storage basins are summarized in Table VII-21.

TABLE VII-21

STORAGE BASIN CAPACITIES AND COSTS

Overflow PT (Fig. VII-19)	Required Volume (MG)	Overflow Basin Volume (MG)	Overflow Basin Construction Cost	O&M $/Yr
1	13.2	13	$7,000,000	$68,000
2	7.7	8	5,100,000	59,000
3	3.0	3	2,800,000	35,000
4	4.9	5	3,943,000	40,000
5	4.0	4	3,350,000	38,000
	32.8	33	$22,193,000	$240,000

TABLE VII-17

CONSTRUCTION COSTS
FOR COMBINED OVERFLOW STORAGE BASINS

2MG BASIN

(1) Retention Basin 135′ X 135′ X 15′				
a) Excav: $140 \times 140 \times 20 \times \frac{1}{27}$				
= 14600 CY @ $4/CY	=		$59,000	
b) Conc:				
Base - $140 \times 140 \times 1.5 \times \frac{1}{27}$				
= 1100 CY @ 175	=	193,000		
Roof - 2/3 X 1100 @ 200	=	146,000		
Walls- $8 \times 140 \times 20 \times 1.5 \times \frac{1}{27}$				
- 1,250 CY @ 200	=	250,000		
Misc. Conc. 400 CY @ 200		80,000		
			669,000	
c) Piling				
200 Piles @ 50′ = 10,000 VF				
10,000 VF @ $15.00	=		150,000	
d) Mechanical Equipment	=		300,000	
				= $1,178,000
(2) Pump Station				160,000
(3) Chlorination Equipment				120,000
(4) Preliminary Treatment				200,000
(5) Sitework, etc.				50,000
				$1,708,000
			30% Engr. & Cost	512,000
			Total =	$2,220,000

TABLE VII-18

CONSTRUCTION COSTS
FOR COMBINED OVERFLOW STORAGE BASINS

5MG BASIN

(1) Retention Basin

a) Excav: $215' \times 215' \times 20' \times \frac{1}{27}$

= 34,250 @ $4/CY — $137,000

b) Conc:

Base = $215 \times 215 \times 1.5 \times \frac{1}{27}$

= 2600 CY @ 175 = 455,000

Roof = $\frac{2}{3}$ 2600 CY @ 200 = 347,000

Walls = $8 \times 215 \times 20 \times 1.5 \times \frac{1}{27}$

= 1920 CY @ 200 = 384,000

Misc. Conc. -900 CY @ 200 = 180,000

1,366,000

c) Piling

500 Piles @ 50' = 25,000 VF
25,000 VF @ $15/VF = 375,000

d) Mechanical Equipment - Aerators — 375,000

2,253,000

(2)	Pump Station	300,000
(3)	Chlorination Equipment	150,000
(4)	Preliminary Treatment	250,000
(5)	Site Work, etc.	80,000
		3,033,000
	30% Engr. & Cost	910,000
	TOTAL =	$3,943,000

TABLE VII-19

CONSTRUCTION COSTS
FOR COMBINED OVERFLOW STORAGE BASINS

7 MG BASIN

(1) Retention Basin

a) Excav.: $250' \times 250 \times 20 \times \frac{1}{27}$

$= 46{,}300$ CY @ \$4/CY — \$185,000

b) Conc:

Base $= 250' \times 250' \times 1.5 \times \frac{1}{27}$

$= 3500$ CY @ \$175 $= 612{,}500$

Roof $= 250' \times 250' \times 1.0 \times \frac{1}{27}$

$= 2315$ CY @ 200 $= 463{,}000$

Walls $= 8 \times 250 \times 20 \times 1.5 \times \frac{1}{27}$

$= 2225$ CY @ 200 $= 445{,}000$

Misc. Conc. -1000 CY @ 250 $= 250{,}000$

1,770,500

c) Piling

600 Piles @ 50' = 30,000 VF
30,000 VF @ \$15.00 VF — 450,000

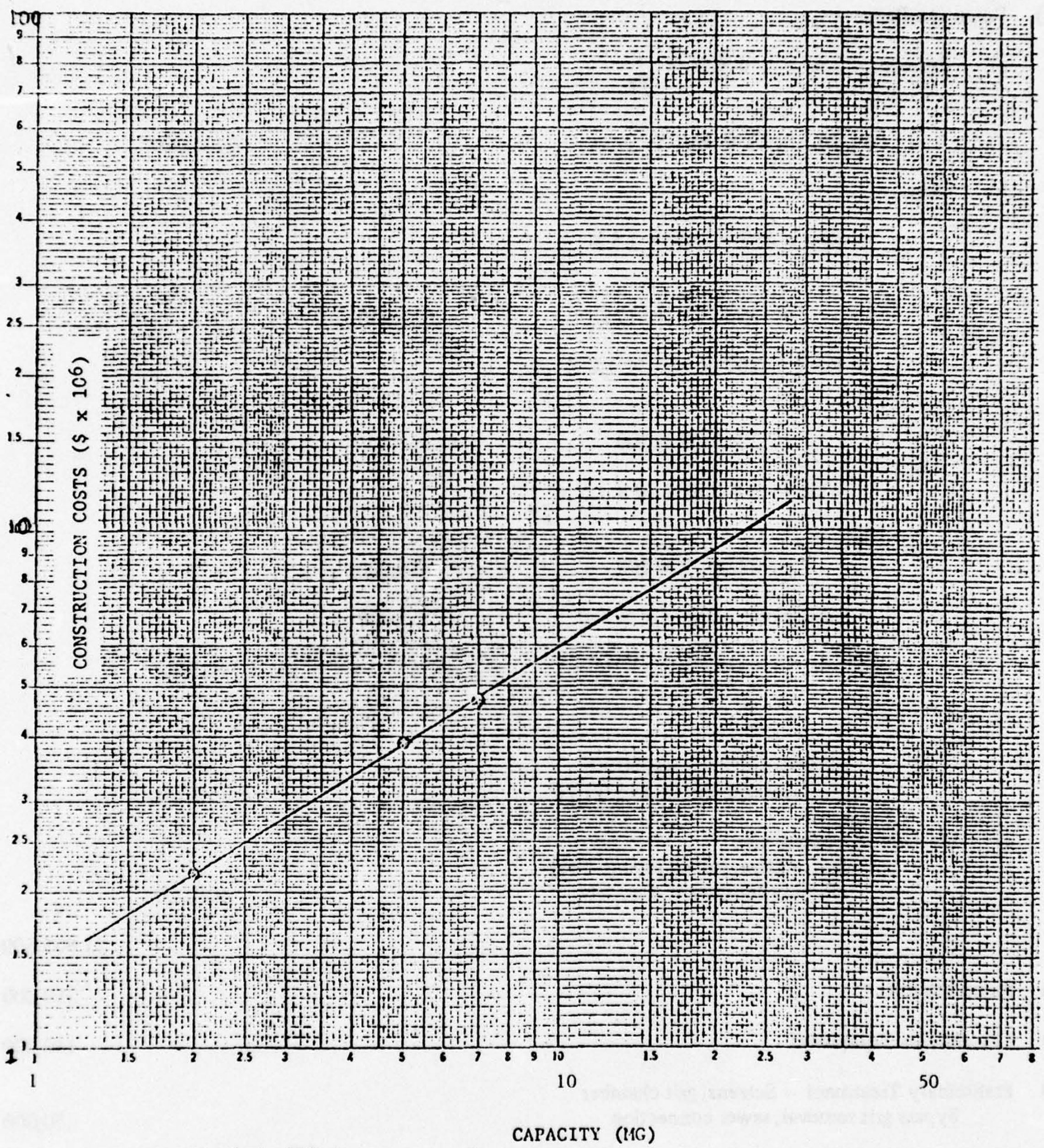
d) Mechanical Equipment
Sluice gates, valves, piping, spray cleaning equipment, fans, etc = 400,000

	\$2,805,500
(2) Pump Station	350,000
(3) Chlorination Equipment	150,000
(4) Preliminary Treatment – Screens, grit chamber bypass grit removal, sewer connection	250,000
(5) Site Work, etc.	100,000
	3,655,500
30% Engr. & Cost	1,097,000
TOTAL=	\$4,752,500

COMBINED OVERFLOW BASIN COSTS

(Dec. 1974)

FIGURE VII-12



costs include 30% for engr. and conting.

TABLE VII-20

OPERATIONS AND MAINTENANCE COSTS FOR STORAGE BASINS OF VARYING CAPACITIES

Overflow Basins	13 mgd	8 mgd	5 mgd
Labor	$10,000	$15,000	$ 5,000
Chemicals	8,000	6,000	4,000
Power & Fuel	6,000	4,000	3,000
General Repairs	14,000	10,000	8,000
Solid Handling	30,000	24 000	20,000
TOTALS	$68,000	$59,000	$40,000

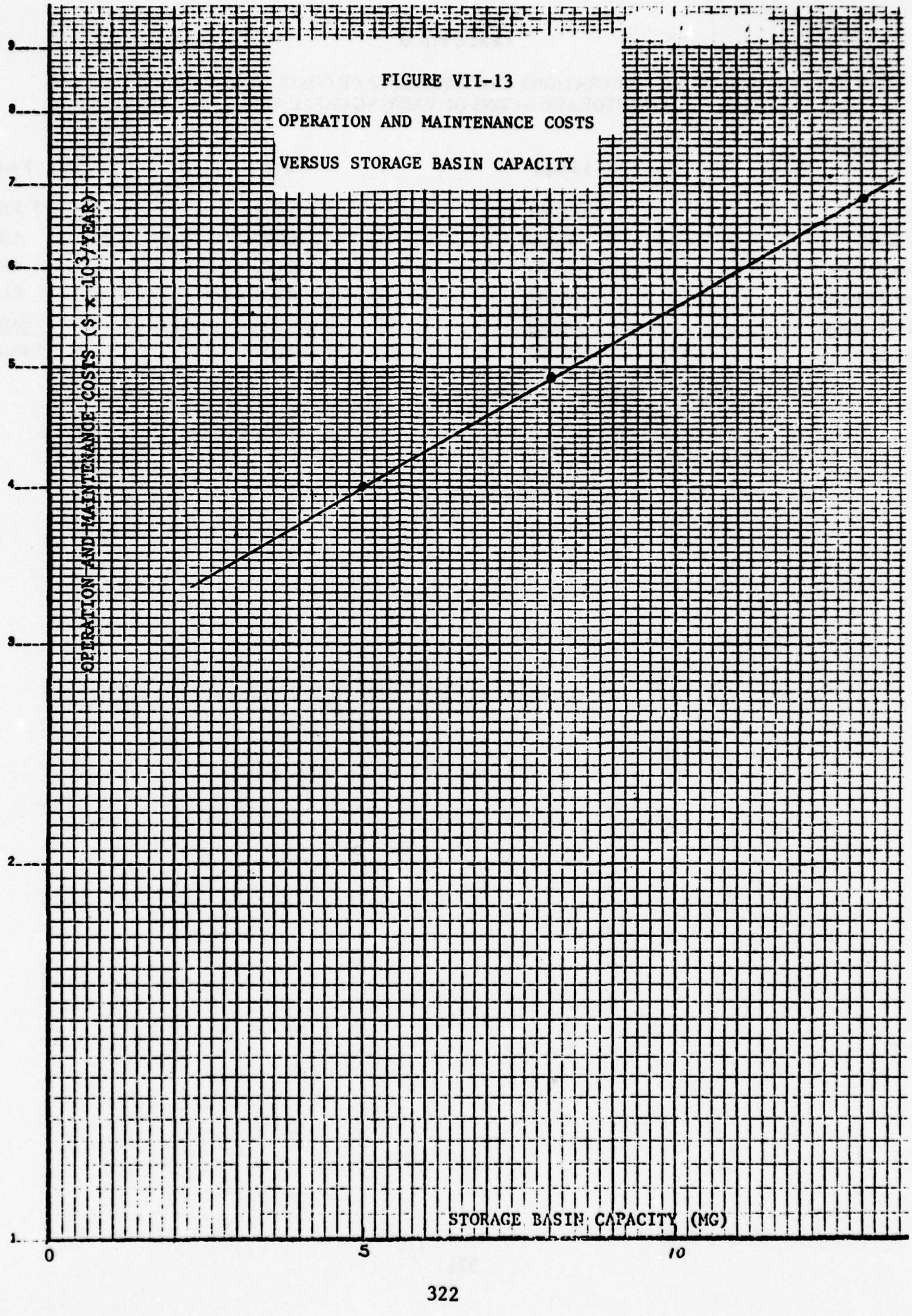


FIGURE VII-13
OPERATION AND MAINTENANCE COSTS
VERSUS STORAGE BASIN CAPACITY
OPERATION AND MAINTENANCE COSTS ($ x 10^3/YEAR)
STORAGE BASIN CAPACITY (MG)
9
8
7
6
5
4
3
2
1
0
5
10

Alternative B--Treatment at Overflow Sites Using Micro-Strainers

Main Characteristics.

Micro-strainers, or microscreens, are essentially mechanical filters on a rotating drum. The filter is usually a tightly woven wire mesh fabric fitted on the drum periphery. The drum is placed in a tank and wastewater flows through the rotating screen on the drum. The filtered solids are continuously backwashed off the screen and removed from the inside of the drum. A schematic of a typical micro-strainer is shown in Figure VII-14. Five treatment facilities, ranging in capacity from 6.5 to 10 mgd would be located at the sites shown in Figure VII-19. A flow diagram for the micro-strainer suggested in this alternative is shown in Figure VII-15. Bar screens would provide preliminary treatment before the overflow is pumped to the micro-screens. A chlorination facility is provided to disinfect the effluent before discharge to the rivers.

Each treatment facility is designed to treat the overflows resulting from the design storm maximum hourly intensity of 0.22 in/hr. The hydrographs at each overflow site were estimated using the SWMM. These hydrographs were the basis for defining the design flow for each microstrainer facility.

Effectiveness.

The minimum instantaneous DO level during the design storm (1.25 inch/ day) would be between 4 and 5 mg/l in conjunction with secondary or improved secondary treatment of wastewater. In conjunction with AWT or land treatment systems, the minimum DO level would be 5 to 6 mg/l.

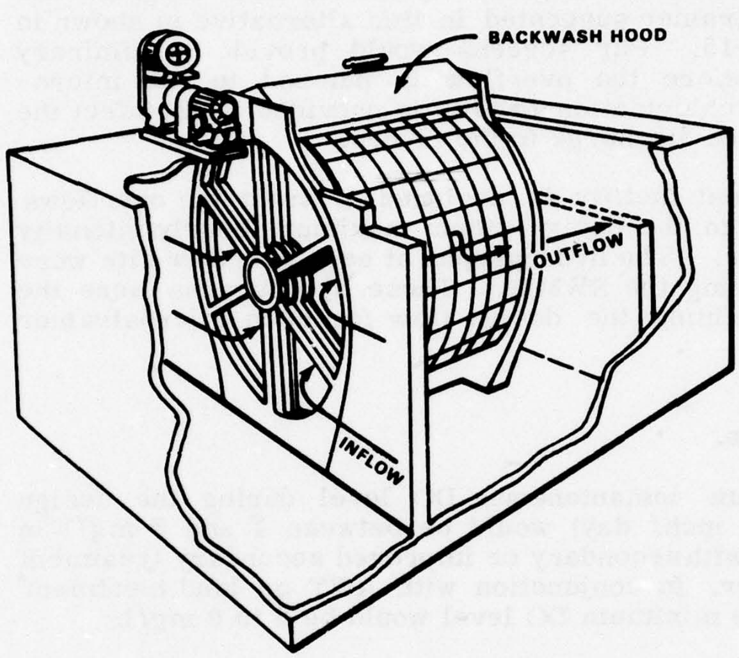
Reduction of pollution loads due to combined sewer overflows are as follows:

a. Short-Term (reduction per storm):

BOD Load Reduction = 4,700 lbs.

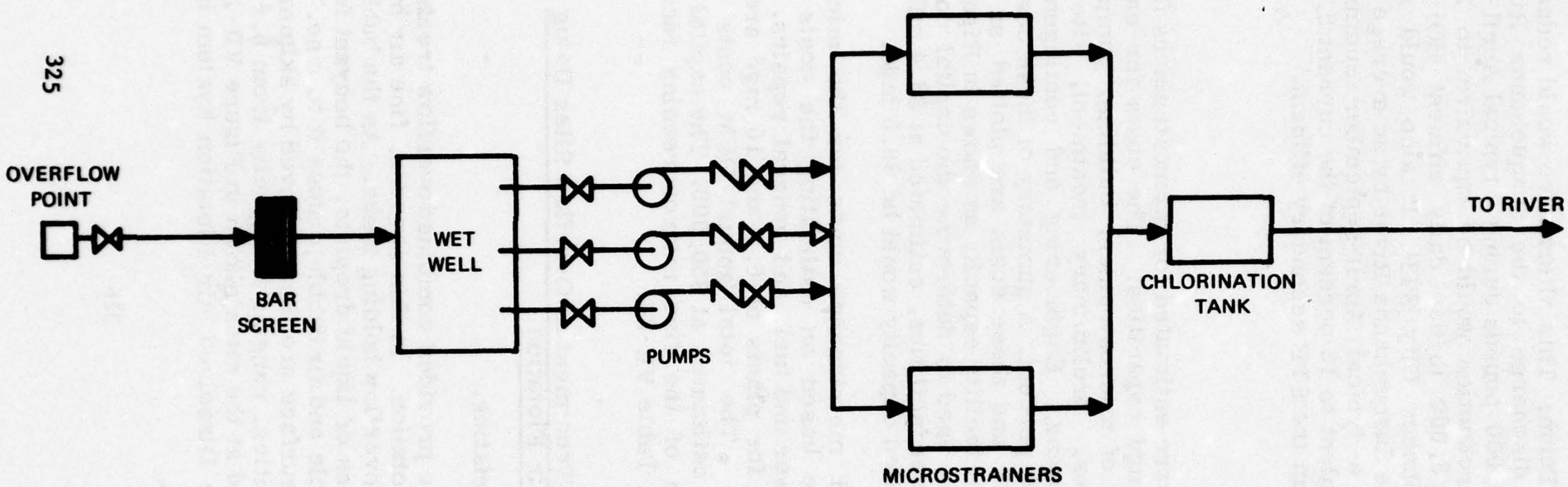
NOD Load Reduction = 700 lbs.

Coliform--Reduce effluent total count to 15,000/100 ml and Susquehanna River total count to slightly more than 1000/100 ml.



SCHEMATIC OF A MICROSTRAINER

FIGURE VII-14



OVERFLOW TREATMENT
MICROSTRAINER SYSTEM

FIGURE VII-15

b. Long Term: This alternative would reduce the BOD pollution load discharge to the Susquehanna River by an average of 125,000 pounds during a typical April-September period. This reduction would be equivalent to 23 percent of the current 3,000 lb/day daily effluent BOD from the Binghamton-Johnson City STP. It also would reduce the NOD load to the Susquehanna River by an average of 125,000 pounds during a typical April-September summer period, which is equivalent to 13 percent of the current 5,200 lb/day discharged from the STP secondary effluent.

Cost Analysis

Capital costs were estimated for microstrainers facilities of 5.0 and 13.0 mgd capacities. The costs for each facility included costs of microstrainer, building, pumps, chlorination facilities, preliminary treatment, sitework, and interceptor piping. Engineering and contingencies were estimated at 30 percent. A summary of these costs is given in Table VII-22 and these costs are plotted as a function of the treatment facility capacity as shown in Figure VII-16. This figure was used to determine the capital costs for all the microstrainer facilities, estimated at $3.4 million total. The total installed capacity would be 39.5 mgd.

Operation and maintenance costs for the microstrainer facilities were based on estimating the costs of labor, chemicals, power and fuel, and general repairs. The O&M cost estimates for plants of 6.5 and 10 mgd are shown in Table VII-23. The total annual O&M costs of all the facilities were estimated at $50,000. The capital plus O&M costs of each of the five microscreening facilities are summarized in Table VII-24.

Alternative C: Treatment at Overflow Sites Using Dissolved Air Flotation

Main Characteristics.

This alternative provides combined overflow treatment using dissolved air flotation. In this process, fine air bubbles are forced into the overflow holding tanks. As the bubbles attach to solid particles or liquid droplets, the bouyant force of the combined particle and air bubble causes it to rise. Particles floating to the surface are then removed by skimming. Five treatment facilities, ranging in capacity from 6.6 to 10 mgd would be located at the sites shown in Figure VII-19. A flow diagram of the Dissolved Air Flotation System is shown in Figure VII-17.

TABLE VII-22

ESTIMATE OF THE CAPITAL COSTS OF MICROSTRAINER FACILITIES

1. **5.0 MGD**

a)	10′ X 10′ microstrainers	$100,000		
b)	Building	30,000		
c)	Pumps & mechanics/piping	40,000		
d)	Chlorination	50,000		
e)	Prelim. treatment	25,000		
f)	Sitework	30,000		
g)	Land - 1 acre @ $15,000/ac.	15,000		
h)	Misc.	35,000		
i)	Interceptor piping & chamber	70,000	Cost	$395,000
	Engineering Contingencies	30%		120,000
				$515,000

2. **13.0 MGD**

a)	2 - 10′ X 10′ microstrainers	200,000	
b)	Building	56,000	
c)	Pumps & mechanical	50,000	
d)	Chlorination	60,000	
e)	Prelim. treatment	30,000	
f)	Sitework, Roads, etc.	70,000	
g)	Land - 1.5 acres @ $15,000/ac.	23,000	
h)	Misc.	67,000	
i)	Int. piping & chamber	210,000	$766,000
	Engineering Contingencies	30%	230,000
			$996,000

FIGURE VII-16

CAPITAL COST
OF STORM OVERFLOW TREATMENT PLANT
USING MICROSTRAINERS/AIR FLOTATION

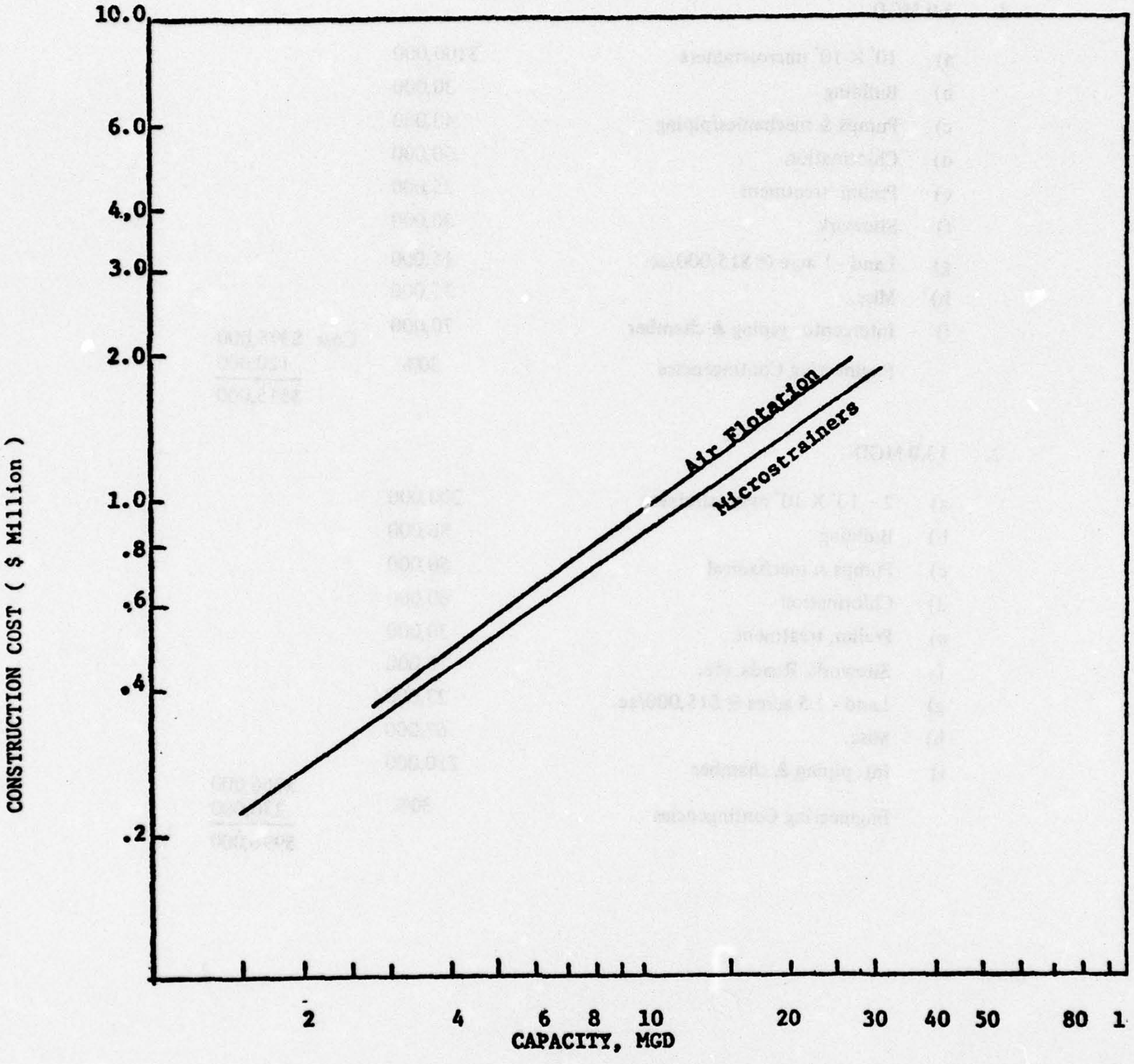


TABLE VII-23

OPERATIONS AND MAINTENANCE COSTS

MICROSTRAINERS FACILITIES

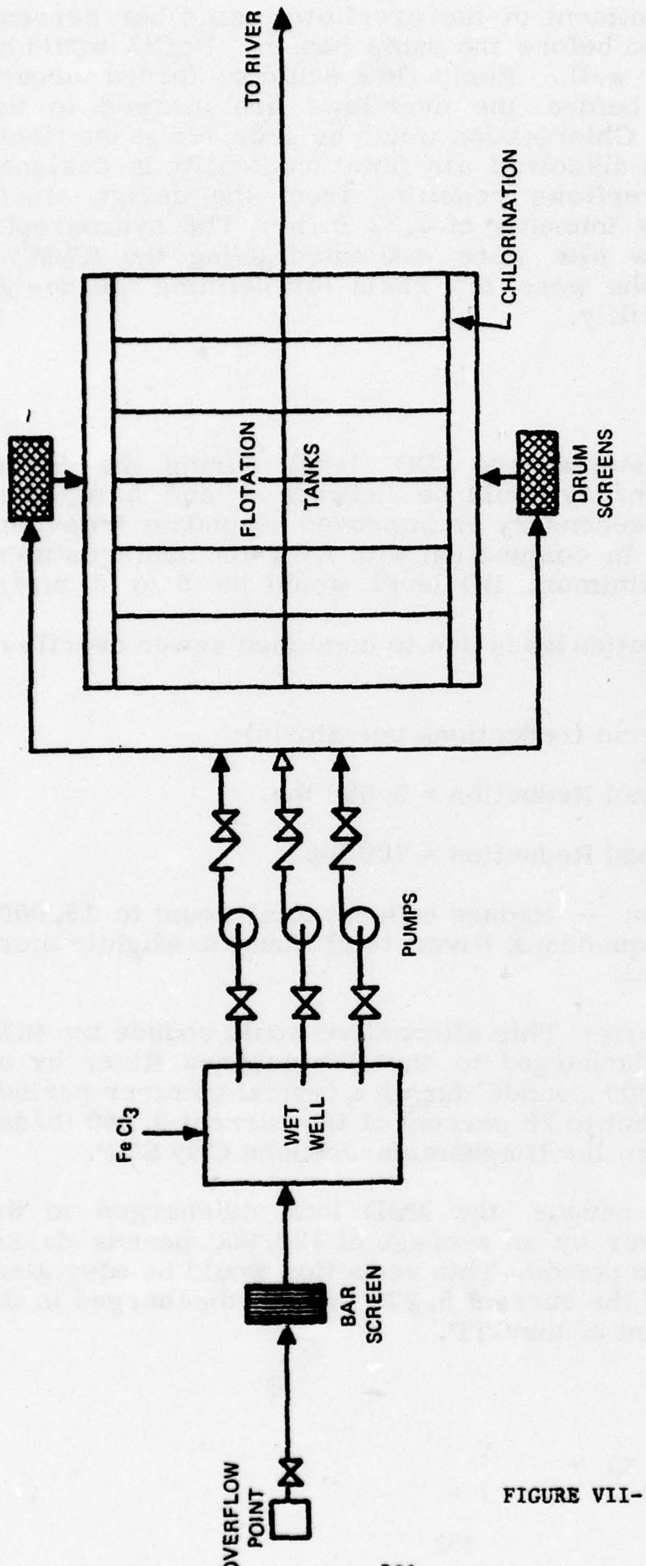
(1)	**15 CFS (10MGD) PLANT**	
	Labor	$ 2,000
	Chemicals	5,000
	Power & Fuel	1,000
	General Repairs	5,000
	TOTAL	$13,000/yr
(2)	**10 CFS (6.5 MGD) PLANT**	
	Labor	$ 1,000
	Chemicals	3,000
	Power & Fuel	800
	General Repairs	3,200
	TOTAL	$ 8,000/yr

TABLE VII-24

CAPACITY, CAPITAL AND O&M COST ESTIMATES OF MICROSTRAINER FACILITIES

Alternate B

Overflow Point (Figure VII-19)	FLOW CFS	FLOW MGD	CONSTRUCTION COST ($)	O&M ($/Year)
1	15	10	$ 800,000	$13,000
2	10	6.5	600,000	8,000
3	15	10	800,000	13,000
4	10	6.5	600,000	8,000
5	10	6.5	600,000	8,000
TOTAL	60	39.5	$3,400,000	$50,000



OVERFLOW TREATMENT
AIR FLOTATION SYSTEM

FIGURE VII-17

Preliminary treatment of the overflows using bar screens would be provided before the pump house. FeCl3 would be added to the wet well. Pump flow would be forced through drum screens before the overflows are allowed to the flotation tanks. Chlorination would be provided at the flotation tanks. Each dissolved air flotation facility is designed to handle the overflows resulting from the design storm maximum hourly intensity of 0.22 in/hr. The hydrographs at each overflow site were estimated using the SWMM. These hydrographs were the basis for defining the design flow for each facility.

Effectiveness.

The minimum instantaneous DO level during the design storm (1.25 inch/day would be between 4 and 5 mg/l in conjunction with secondary or improved secondary treatment of wastewater. In conjunction with AWT or land treatment systems, the minimum DO level would be 5 to 6 mg/l.

Reduction of pollution loads due to combined sewer overflows are as follows:

a. Short Term (reductions per storm):

BOD Load Reduction = 5,600 lbs.

NOD Load Reduction = 700 lbs.

Coliform -- Reduce effluent total count to 15,000/100 ml, and Susquehanna River total count to slightly more than 1,000/100 ml.

b. Long Term: This alternative would reduce the BOD pollution load discharged to the Susquehanna River by an average of 150,000 pounds during a typical summer period, which is equivalent to 28 percent of the current 3,000 lb/day effluent BOD from the Binghamton-Johnson City STP.

It also would reduce the NOD load discharged to the Susquehanna River by an average of 125,000 pounds during a typical summer period. This reduction would be equivalent to 13 percent of the current 5,200 lbs/day discharged in the secondary effluent of the STP.

Cost Analysis.

Capital costs were estimated for dissolved air flotation treatment facilities having capacities of 5.0 and 13.0 mgd. The cost for each facility included the costs of the screening/flotation system, the control building, interceptor piping, and chamber and sitework. Engineering and contingencies were estimated at 30 percent.

A summary of the costs for 5.0 mgd and 13 mgd air flotation facilities are shown in Table VII-25. Their costs were plotted as a function of the storage basin capacity in Figure VII-16. This figure was used to estimate the capital costs for this alternative, estimated to total \$4.0 million. The total installed capacity would be 39.5 mgd.

Operations and maintenance costs for the dissolved air flotation facilities were based on estimating the costs of labor, chemicals, power, and general repairs. The O&M costs estimates for 6.5 and 10 mgd plants are shown in Table VII-26.

The annual O&M costs of all the facilities were estimated at \$77,000. The capital plus O&M costs of each of the five dissolved air flotation facilities are presented in Table VII-27.

Alternative D: Centralized Treatment of Combined Overflows Using Modified Biological Treatment

Main Characteristics.

A system of interceptors would collect the overflows to a central treatment facility to be built either on the existing Binghamton-Johnson City STP site or in one of the two sites shown in Figure VII-18. The facility would be a modification of the contact stabilization type activated sludge process (secondary treatment). Provided the treatment plant site has enough vacant land, it would be advantageous to build the facility at the existing STP to achieve dual use of the facility during dry and wet weather conditions. Also, a saving in O&M costs could be achieved this way. If an upgraded waste treatment facility were to be built on the available vacant land, then Site #2 would be the next desirable location for the combined sewer overflow treatment facility.

The new interceptors included in this system would replace some of the existing trunk and interceptor lines that are old

TABLE VII-25

ESTIMATES OF THE CAPITAL COSTS OF DISSOLVED AIR FLOTATION SYSTEMS

5.0 MGD

1)	Screening/ flotation sys., pumps	=	$ 250,000
2)	Control building, Roof over flotation tank	=	30,000
3)	Sitework	=	30,000
4)	Misc.	=	50,000
5)	Interceptor piping & chamber	=	70,000
			430,000
	Engineering & Contingencies - 30%		129,000
			$ 559,000

13 MGD

1)	Screening/flotation system pumps	=	$ 650,000
2)	Control building, Roof	=	40,000
3)	Sitework	=	70,000
4)	Misc.	=	90,000
5)	Interceptor piping & chamber	=	210,000
			1,060,000
	Engineering & Contingencies - 30%		318,000
			$1,378,000

TABLE VII-26

ESTIMATE OF OPERATION AND MAINTENANCE COSTS FOR DISSOLVED AIR FLOTATION SYSTEMS

6.5 MGD Plant

Labor	$ 1,000
Chemicals	5,000
Power	800
General Repairs	6,000
Total Annual Costs	$12,800

10 MGD Plant

Labor	$ 2,000
Chemicals	7,000
Power	1,200
General Repairs	9,000
Total Annual Cost	$19,200

TABLE VII-27

CAPACITY, CAPITAL AND O&M COST ESTIMATES OF AIR FLOTATION FACILITIES

Alternative C

Overflow Point (Figure VII-19)	FLOW CFS	FLOW MGD	CONSTRUCTION COST	O&M $/Year
2	15	10	$ 950,000	$19,000
4	10	6.5	700,000	13,000
8	15	10	950,000	19,000
11	10	6.5	700,000	13,000
15	10	6.5	700,000	13,000
TOTAL	60	39.5	$4,000,000	$77,000

BINGHAMTON – JOHNSON CITY STP EXPANSIONS AND CORRESPONDING SEWAGE TRANSPORT SYSTEMS

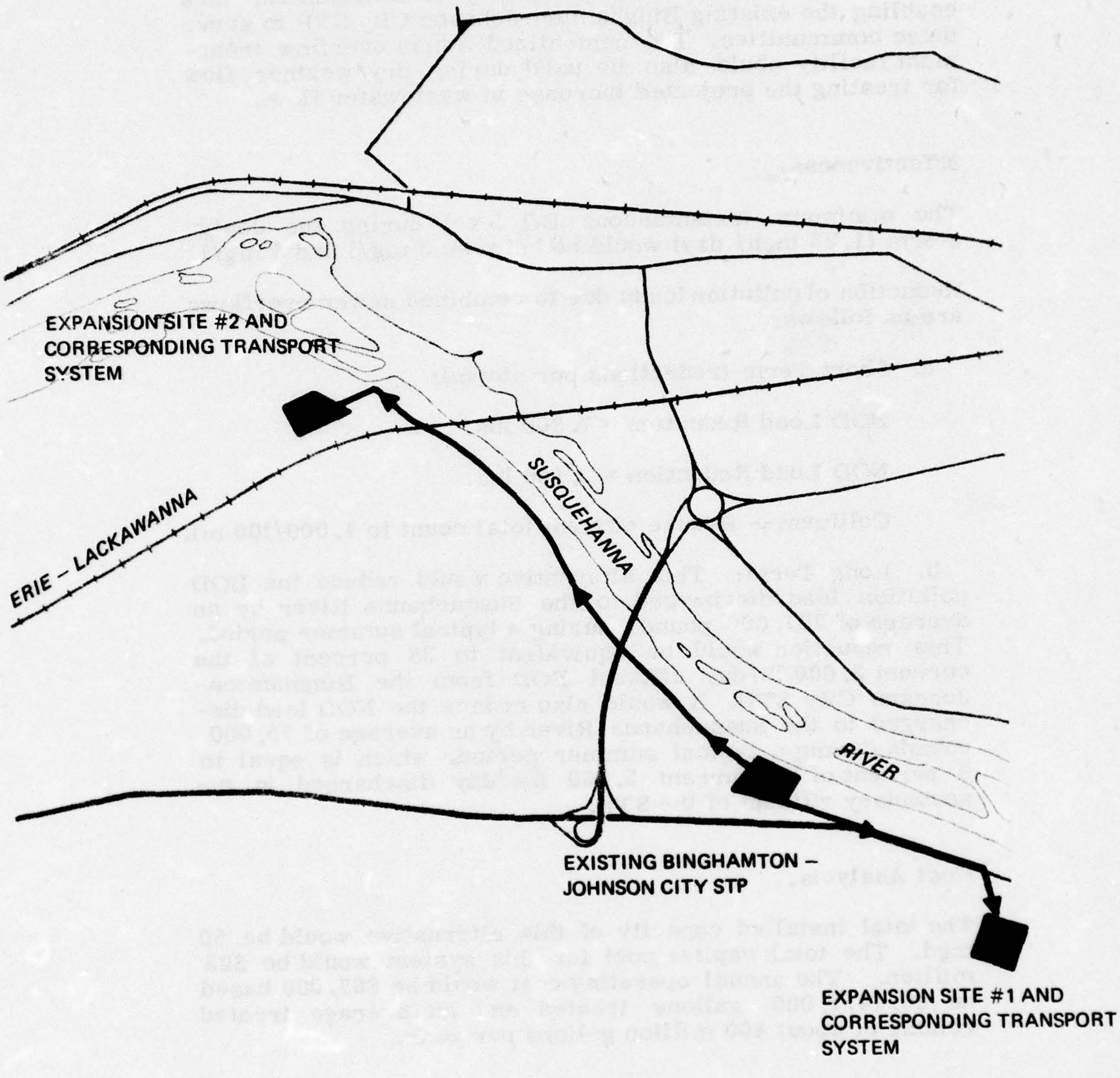


FIGURE VII-18

and subject to high infiltration rates. This could prove effective in controlling significant amounts of infiltration. Interceptor capacity would be designed to handle the combined overflows from a 1.25 inch rainfall with a maximum hourly intensity of 0.22 inches. These interceptors would also provide enough capacity to handle wastewater flows from the communities surrounding the City of Binghamton, thus enabling the existing Binghamton-Johnson City STP to serve these communities. The centralized storm overflow treatment facility could also be used during dry weather flow for treating the projected increase in wastewater flows.

Effectiveness.

The minimum instantaneous DO level during the design storm (1.25 inch/ day) would be between 5 mg/l and 6 mg/l.

Reduction of pollution loads due to combined sewer overflows are as follows:

a. Short Term (reductions per storm):

BOD Load Reduction = 7,800 lbs.

NOD Load Reduction = 2,100 lbs.

Coliform -- Reduce effluent total count to 1,000/100 ml.

b. Long Term: This alternative would reduce the BOD pollution load discharged to the Susquehanna River by an average of 207,000 pounds during a typical summer period. This reduction would be equivalent to 38 percent of the current 3,000 lb/day effluent BOD from the Binghamton-Johnson City STP. It would also reduce the NOD load discharged to the Susquehanna River by an average of 75,000 pounds during a typical summer period, which is equal to 8 percent of the current 5,200 lbs/day discharged in the secondary effluent of the STP.

Cost Analysis.

The total installed capacity of this alternative would be 50 mgd. The total capital cost for this system would be $23 million. The annual operating cost would be $63,000 based on $0.14/1,000 gallons treated and an average treated volume of about 400 million gallons per year.

Summary of Costs

Summarized in Table VII-28 are the capital costs, O&M costs, and total average annual cost for each of the four storm management alternatives. Capital and O&M costs were estimated for the facilities required for each storm control alternative. Estimates were based on January 1975 cost figures. Annual costs were based on an interest rate of 6 1/8 percent for 50 years and an equivalent service life (replacement) of facilities for 27 years. Explanation of the selection of this equivalent life is given in Chapter II.

To allow expression of storm overflow treatment costs as domestic and industrial sources costs, unit costs per million gallons of flow treated and thousand pounds of BOD removal were calculated. The annual cost per capita of controlling combined overflow was also calculated. This will assist in evaluating the social and economic impact of the costs of storm control facilities. A detailed description of the socio-economic impacts of the various stormwater management systems is given in the Impact Assessment and Evaluation Appendix.

SITE INVESTIGATION FOR LOCATING COMBINED OVERFLOW FACILITIES

As previously discussed and shown in Figure VII-7, 21 sewer surcharge points were identified by the SWMM model. Photographic maps of the City of Binghamton were then examined to define potential sites for locating combined overflow control facilities. Because of extensive development within Binghamton, sites for such facilities were quite limited. Site visits and preliminary investigations were conducted to define locations with sufficient area for an overflow control structure.

Stormwater control for the combined sewer system in the City was found to be most feasible at the five sites depicted in Figure VII-19. The two major considerations for the sites finally selected were their proximity to an overflow point (based on SWMM predictions) and the fact the adjacent land was presently vacant. No investigations were made as to the availability of these sites (i.e., ownership, land value, or any planned development by the owners). The control facilities at these five locations would prevent all overflow occurrences from the Binghamton sewer system up to the design storm of 1.25 inches per day and 0.22 inches per hour.

TABLE VII-28

SUMMARY OF COSTS OF STORMWATER MANAGEMENT ALTERNATIVES FOR THE CITY OF BINGHAMTON[1]

Cost	Alternative A Storage	Alternative B Microstrainers	Alternative C Dissolved Air Flotation	Alternative D Centralized Treatment
Capital ($)	22,200,000	3,400,000	4,000,000	23,000,000
O & M ($/year)	240,000	50,000	77,000	63,000
Average Annual Cost ($/year)[2]	1,930,000	311,000	386,000	1,888,000
Annual Capital Cost per 1,000 Gallons Treated ($/1,000 gal.)	4.20	0.70	0.87	4.26
Average Cost per 1,000 lbs BOD Removal ($/1,000 lbs)	8.50	2.49	2.57	9.11
Total Annual Costs/ Capita ($/capita)[3]	30	4.87	6.00	29

[1] January 1975 prices.
[2] 50 years @ 6-1/8%; and includes capital and O & M costs and replacement after 27 years.
[3] Based on current City population of 64,000.

CITY OF BINGHAMTON
LOCATION OF PLANNED OVERFLOW CONTROL FACILITIES

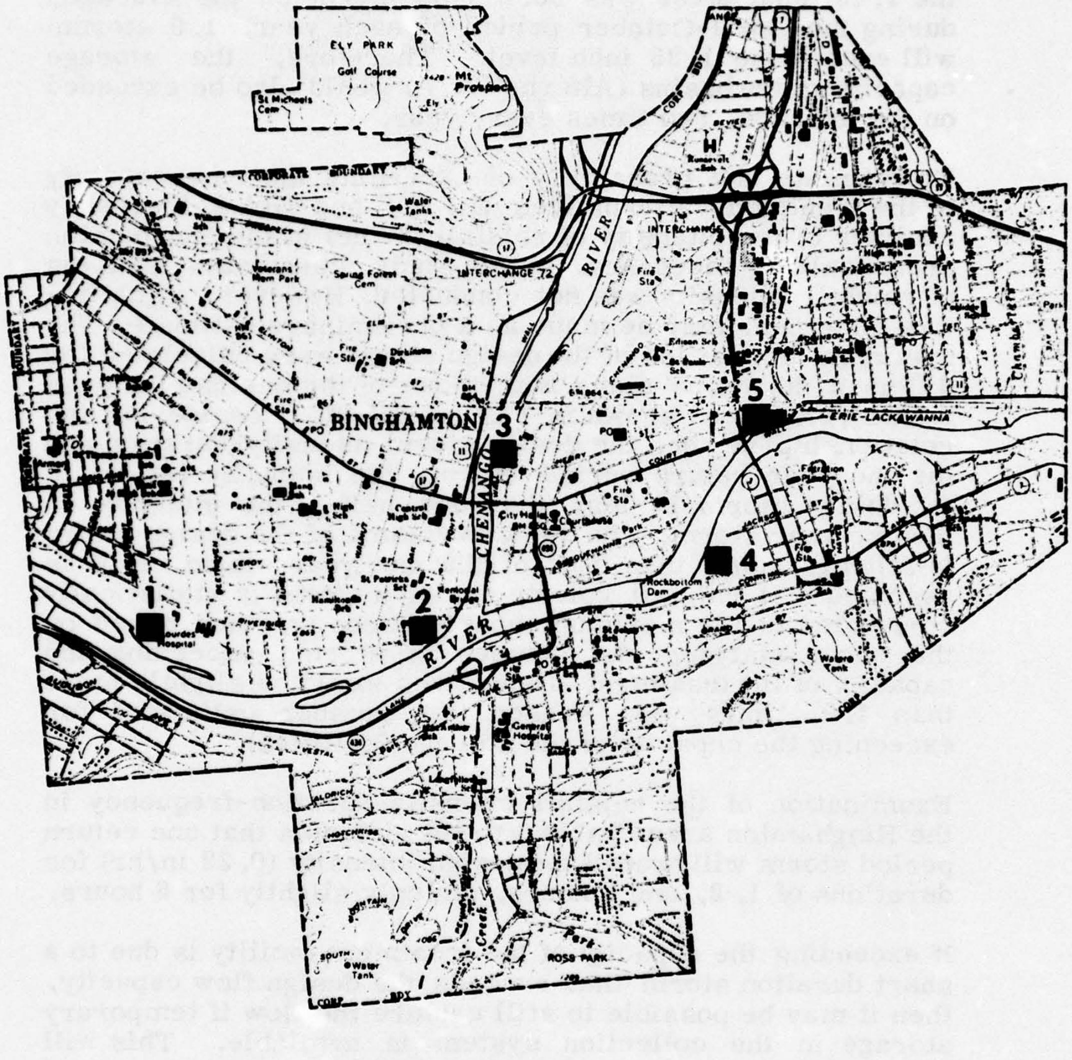


FIGURE VII-19

PROBABILITY OF EXCEEDING THE CAPACITY OF THE STORM OVERFLOW CONTROL ALTERNATIVES

The design storm was a 1.25 inch storm in a 24 hour period. Considering the data in Table VII-7, during the 30 year period of analysis, the total number of rainfall events that exceeded the 1.25 inch level was 58. This meant on the average, during the April-October period of each year, 1.9 storms will exceed the 1.25 inch level. Therefore, the storage capacity of the basins (Alternative A) would also be exceeded on the average, two times every year.

To estimate the probability of exceeding the rate capacity of the treatment alternatives (B, C, and D), a frequency analysis of the intensity of rainfall (in/hr) in the Binghamton area would be necessary. In this Study, correlation analysis of rainfall intensity was not conducted. However, a qualitative statement may be made as a preliminary estimate. The maximum intensities of the design storm were calculated for different durations along with those of the one and ten year return periods as shown in Figure VII-20. Considering the relationship between the design storm intensity-duration and the one year return period storm, the intensity (in/hr) of the design storm is less than one half of the intensity of the one year storm. The 24 hour volume of the design storm is slightly more than half of the one year storm. If it is assumed that rainfall volume during a storm is statistically correlated to the maximum hourly intensity, then based on the above analyses, the number of storms exceeding the capacity of the treatment alternatives would be slightly more than 1.9 times per season (the number estimated for exceeding the capacity of the storage facilities).

Examination of the rainfall intensity-duration-frequency in the Binghamton area (Table VII-10) indicates that one return period storm will exceed the design intensity (0.22 in/hr) for durations of 1, 2, and 3 hours, and only slightly for 6 hours.

If exceeding the capacity of the treatment facility is due to a short duration storm that exceeds the design flow capacity, then it may be possible to still capture the flow if temporary storage in the collection system is available. This will reduce the number of capacity exceedances. It was assumed that before the final design of treatment facilities is made, a detailed statistical analysis of rainfall intensity will be conducted to enable a more accurate estimate of the probability of exceeding the capacity of a given treatment facility. It should be noted that exceeding the capacity of any alternative would result in the discharge of untreated

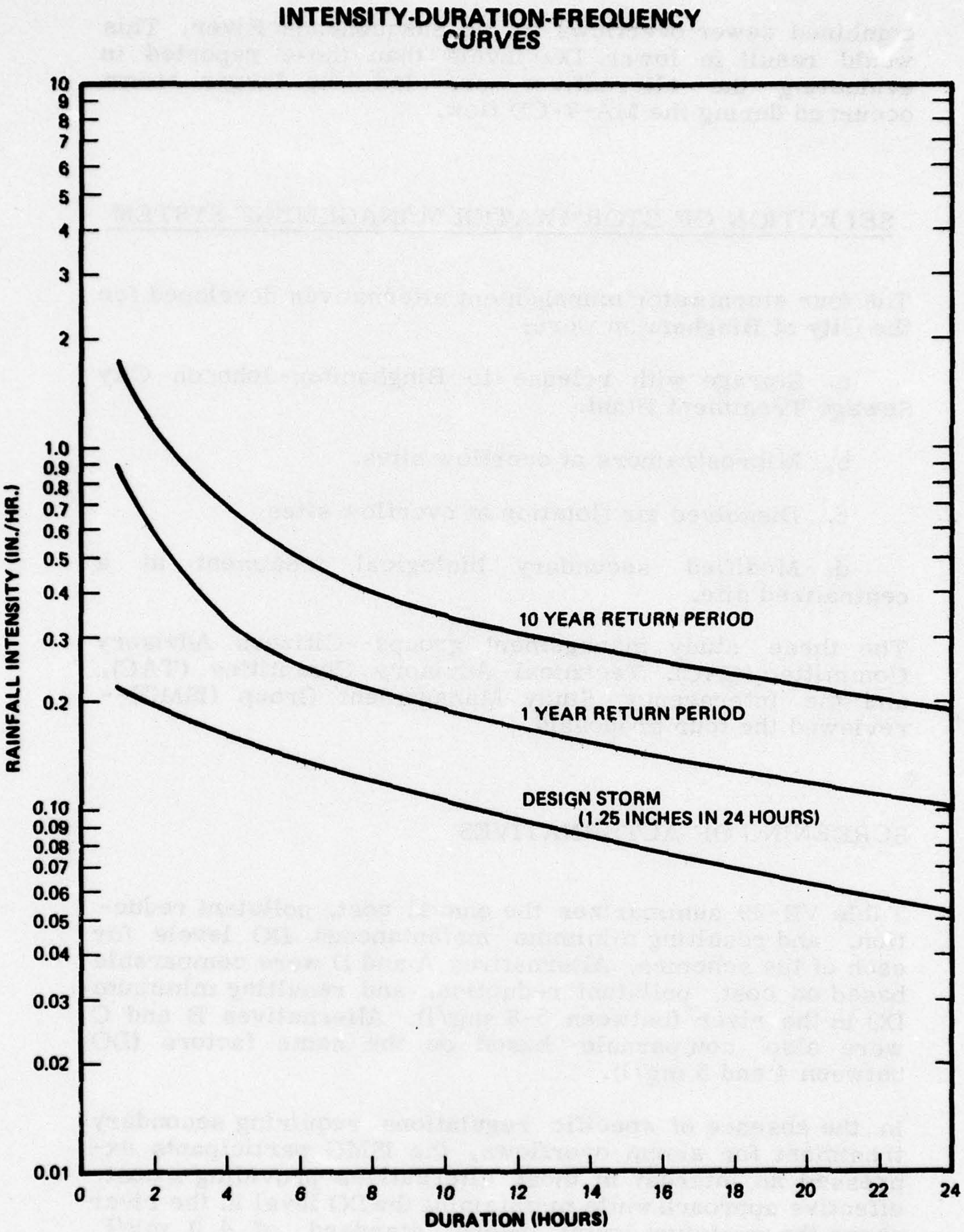


FIGURE VII-20

combined sewer overflows to the Susquehanna River. This would result in lower DO levels than those reported in evaluating the alternatives provided the larger storm occurred during the MA-7-CD flow.

SELECTION OF STORMWATER MANAGEMENT SYSTEM

The four stormwater management alternatives developed for the City of Binghamton were:

a. Storage with release to Binghamton-Johnson City Sewage Treatment Plant.

b. Microstrainers at overflow sites.

c. Dissolved air flotation at overflow sites.

d. Modified secondary biological treatment at a centralized site.

The three study management groups--Citizens Advisory Committee (CAC), Technical Advisory Committee (TAC), and the Interagency Study Management Group (ISMG)--reviewed the four proposals.

SCREENING OF ALTERNATIVES

Table VII-29 summarizes the annual cost, pollutant reduction, and resulting minimum instantaneous DO levels for each of the schemes. Alternatives A and D were comparable based on cost, pollutant reduction, and resulting minimum DO in the river (between 5-6 mg/l). Alternatives B and C were also comparable based on the same factors (DO between 4 and 5 mg/l).

In the absence of specific regulations requiring secondary treatment for storm overflows, the ISMG participants expressed an interest in those alternatives providing a cost-effective approach while maintaining the DO level in the river above the minimum instantaneous standard of 4.0 mg/l. Therefore, the high costs of Alternatives A and D, although achieving 5-6 mg/l DO, were not attractive. Additionally, both of these schemes would involve major construction activities.

TABLE VII-29

ANNUAL COST AND EFFECTIVENESS
STORMWATER MANAGEMENT ALTERNATIVES
CITY OF BINGHAMTON
(Based on 1.25 inch storm in 24 hours)

		Alternatives			
		A	B	C	D
Effect	No Action	Storage	Micro-strainers	Dissolved Air Flotation	Centralized Treatment
Average Annual Cost ($/Yr)	0	1,930,000	311,000	386,000	1,888,000
Overflow Load					
BOD (lb)	9,400	940	4,700	3,800	1,600
% Reduced	0	90	50	60	83
NOD (lb)	4,200	2,100	3,500	3,500	2,100
% Reduced	0	50	17	17	50
Minimum Instantaneous River DO (mg/l)	3-4	5-6	4-5	4-5	5-6
Coliform in Susquehanna River (MPN/100 ml)	240,000	1,000	1,000	1,000	1,000

The management groups reached a general consensus that on-site treatment with either Alternative B, Microstrainers, or C, Dissolved Air Flotation, would be desirable. In considering the merits of each, Alternative C would remove a slightly higher percentage of BOD, but Alternative B would be easier to operate, would have fewer mechanical problems, and had the least cost.

RECOMMENDATIONS

Based on the recommendations of the other two committees, the ISMG selected Alternative B, utilizing microstrainers and chlorination for treatment of combined sewer overflows. Although the secondary treatment level would not be attainable with the microstrainer process, it was acceptable to the agencies comprising the ISMG as this process could remove a majority of the settleable solids at a minimum of cost and would meet stream standards. Under this alternative, all excess combined sewer flow in the City system would be treated at five overflow sites (see Figure VII-19) prior to discharge to the Susquehanna and Chenango Rivers. Alternative B was recommended as a component to each wastewater management plan developed in Stage III.

CHAPTER VIII

FINAL PLANS

PLANS FOR FINAL ANALYSIS

During Stage III, complete wastewater management plans were developed. These plans integrated the wastewater, sludge and stormwater management components described in the previous chapters. The technical analyses needed to refine and screen the alternatives were conducted to the level enabling the decision makers -- the Citizens Advisory Committee (CAC), Technical Advisory Committee (TAC) and Interagency Study Management Group (ISMG) -- to make a selection of the plan components. This planning process is documented in the *Plan Formulation Appendix.*

The following were the major principles for establishing the proposed plans, based on the overall criteria for water resources planning and on the concerns of the ISMG, CAC, and TAC.

1. A least cost plan to meet water quality standards should be included for final consideration.

2. Plans with a DO concentration of either 4.0 mg/l or 5.0 mg/l (predicted by the stream model) can presently be considered to satisfy principle 1.

3. Both five and six plant regionalization systems should be considered for plans attaining either 4 or 5 mg/l DO. A phased service area and treatment plant for Chenango Valley should also be analyzed.

4. A system meeting or approaching the 1985 zero discharge goal should be considered.

5. Flow reduction by nonstructural means and/or infiltration control should be considered for each plan.

Based on the above guidelines, further analysis led to the conclusion that to maintain 4 mg/l or 5 mg/l DO in the Susquehanna River, at least cost, the most cost-effective regionalization scheme would employ the three existing STP's in Tioga County.

However, the ISMG felt there were some unresolved issues concerning Broome County regionalization, specifically with respect to treatment of Chenango Valley wastes. Under the present cost sharing agreements between Binghamton-Johnson City and Outlying Communities, Chenango Valley residents would actually pay more for regionalization with Binghamton-Johnson City, than subregionalization (separate STP for Chenango Valley) even though the total present worth of regionalization would be slightly less. Regionalization would aggravate existing problems of stormwater overflows and infiltration in the City sewer system and the limited expansion capabilities of the Binghamton-Johnson City STP.

Considering these factors, the ISMG recommended that, in addition to the continued analysis of both the regional and subregional systems, for the Binghamton-Chenango area, a phased plan for subregionalization (separate Chenango plant) be formulated and evaluated. Under this system, sewering would be phased and the initial Chenango Valley service area would be limited with a correspondingly smaller treatment plant. The primary intent of the phased plan would be to attain a faster implementation for sewer service, where severe needs exist, than would be realized by the other schemes which would service all of the Chenango Valley Area initially.

Biological based advanced waste treatment was more cost-effective and more desirable from the standpoint of resource commitments and degree of flexibility than physical/chemical AWT. The optimal degree of regionalization at this level of treatment would be two plants in Broome County (Binghamton-Johnson City and Endicott) and two plants in Tioga County (Town of Owego STP's 1 and 2).

Land application of secondary treated effluent was undesirable due to the large commitment of land. The beneficial impact of land application on the Susquehanna River could be exceeded by Bio AWT with much less adverse impact. There was no compelling factor dictating further consideration of land application.

There were four plans recommended for final plan detail. The level of treatment, amount of infiltration control, degree of regionalization (number of STP's), nonstructural measures, and sludge management, were specifically defined for each plan. These plans are presented in Table VIII-1.

TABLE VIII-1

FINAL PLANS

Plan No.	Description	Year 2020 DO Levels with Daily Pollutant Loadings @ MA-7-CD-10 Flows: Minimum Instantaneous	Minimum Daily Avg.	Number of Plants: Broome Co.	Tioga Co.	Binghamtons Infiltration Control (mgd)	Non-Structural Measures	Institu-tions	Total Presen Worth (Millio $'s)**
1	Baseline	3.0	3.5	2	3	0	None	Existing	31.9
2A	Secondary	3.9	4.4*	2	3	1.0	None	Modified or District	47.2
2B	Secondary	4.0	4.5*	3	3	1.0	None	Modified or District	47.7
2C	Secondary (Phased Service)	4.0	4.5*	3	3	1.0	None	Modified or District	47.7
3A	Secondary + Nitrification	4.9	5.4	2	3	3.0	None	Modified or District	50.5
3B	Secondary + Nitrification	5.0	5.5	3	3	3.0	None	Modified or District	51.1
3C	Secondary + Nitrification	5.0	5.5	3	3	3.0	None	Modified or District	51.1
4	Advanced Waste Treatment	6.2	6.7	2	2	3.0	Pricing & Education	County Dpt or Two-Co. Authority	90.2

* Contrevention of NYSDEC standard for minimum daily average DO (5 mg/l) may occur after 1994.

** Includes all capital, O & M, and replacement costs; 50 year economic life @ 6 1/8%.

The final plans, in addition to the Baseline Condition, include three levels of treatment:

1. Maintain the DO level, (as predicted by the stream model) in the Susquehanna River above 4.0 mg/l.

2. Maintain the DO level above 5.0 mg/l.

3. Approach the 1985 Federal goal of zero discharge of pollutants.

Secondary treatment (Plan 2) would be sufficient to achieve the 4 mg/l DO level. To maintain a minimum of 5 mg/l, nitrification would be added at the Binghamton-Johnson City STP (Plan 3). Advanced Waste Treatment systems (Plan 4) would be applied to all plants to meet or approach the 1985 goal of zero discharge as defined by the Corps of Engineers.

The optimal numbers of STP's varied with the level of treatment. For the AWT system, four plants are cost-effective: Binghamton-Johnson City and Endicott in Broome County, and Town of Owego STP's No. 1 and No. 2 in Tioga County.

For plans achieving 4 or 5 mg/l DO, three plants in Tioga County are optimal: Town of Owego STP's No. 1 and No. 2 and Owego Village STP. For Broome County, the most effective degree of regionalization was uncertain, and at the time, was felt to depend upon the results of a sewer evaluation study for Binghamton. The question centered on whether to treat Chenango Valley wastes (currently not sewered) at the Binghamton-Johnson City plant or to construct a separate facility at Chenango Valley. Three schemes were developed to handle wastewater in Broome County: (1) regionalization--Binghamton-Johnson City and Endicott STP's (Plans 2A and 3A); (2) sub-regionalization--Binghamton-Johnson City and Endicott STP's and a new Chenango Valley STP (Plans 2B and 3B); and (3) sub-regionalization with phased sewering for a smaller initial plant in Chenango Valley (Plans 2C and 3C).

The following additional components were applied to all but the Baseline Plan:

1. Infiltration control for the City of Binghamton at the cost-effective level.

2. Nonstructural flow reduction was considered for all plans and would be at the level found to be cost-effective for each degree of treatment and for each service area.

3. The most efficient sludge management scheme for all plans is the application of liquid sludge to agricultural land, plus a monitoring program.

4. Stormwater management in both Binghamton and Owego Village should be by microscreening and chlorination of combined sewer overflows.

PLAN 1--BASELINE

The 1977 Baseline Plan, essentially a "no action" plan, established the basis for the impact assessment of the "action" plans. This baseline projected the impacts which would result (by year 2020) if the Bicounty Area's wastewater management systems remained unchanged from those presently planned for the year 1977. The plan consisted of a static physical system subjected to projected future conditions, i.e., increased population and wastewater flows.

EXISTING PRIORITIES

In defining the environmental, social, and economic projections for the Baseline Plan, it was assumed no wastewater treatment facilities would be added other than those already approved by New York State Department of Environmental Conservation (NYSDEC) for construction prior to 1977. Expansion of the present service areas was assumed to continue along existing trends. The additional interceptors approved by NYSDEC for funding in 1975 and 1976 were included in the Baseline condition, except for the Chenango Valley interceptor as Broome County was studying other alternatives. Those sewers providing service for growth areas that are included in the Broome County Sewerage Feasibility Plan were also assumed to be in the 1977 Baseline Plan. The locations of treatment plants, interceptors, and service areas for this plan are depicted in Plate 2. The following is a description of the facilities in the Baseline Plan that are ranked by NYSDEC for funding prior to 1977.

Owego Village: An interceptor, pump station, and a force main to serve all areas south of the river and within the village; part of River Road and the Valley View Heights subdivision; the area within the Village lying west of the creek; and the low area lying east of the Court Street Bridge, including Lackawanna Avenue and Route 17. The existing primary treatment plant will be upgraded to provide for secondary treatment. This project is scheduled for funding in 1976.

Town of Union: Extension of sanitary sewers to the Choconut Center area of the Town of Union is scheduled for funding by 1977.

Town of Vestal: An interceptor sewer to the Endicott STP is scheduled for funding by 1977. This interceptor will serve the western portion of the Town of Vestal which is currently served by a primary treatment plant (to be abandoned).

Town of Owego: An interceptor serving the eastern part of the Town and connecting to Owego STP #2 is scheduled for funding by 1977.

MUNICIPAL WASTEWATER MANAGEMENT

Regionalization of STP's

The Broome County plants would be the existing Binghamton-Johnson City and Endicott STP's. The Chenango Valley area would not be sewered. Tioga County plants would be the existing Town of Owego STP's No. 1 (West Owego) and No. 2 (East Owego) and the Village of Owego STP. The Vestal STP would be abandoned with diversion of its sewage via a new interceptor to the Endicott STP. The Owego Valley View Imhoff Tank would be abandoned with diversion of its sewage to an upgraded Owego Village STP. Extensions of sewage collection and treatment services would take place within the Nanticoke Creek Valley (to the Endicott STP) and toward Five Mile Point (to the Binghamton-Johnson City STP). Service areas and interceptors included in this plan are shown on Plate 2.

Treatment Levels and Processes

The Owego Village STP would be upgraded to provide secondary treatment. The remaining STP's which currently provide secondary treatment would not be expanded or upgraded although the sewered population and sewage flows to all STP's would continue to increase. The Chenango Valley area would continue on septic systems.

Infiltration Control Level in the City of Binghamton

Under the Baseline Condition, no control measures would be implemented and infiltration would remain at its current average daily level of about 7.5 mgd.

Nonstructural Measures for Flow Reduction

Nonstructural measures for flow reduction would not be applied under the Baseline Condition.

STORM OVERFLOW MANAGEMENT

Combined sewer overflows in the City of Binghamton and Owego Village systems would continue to discharge untreated sewage and stormwater to the Susquehanna and Chenango Rivers.

SLUDGE MANAGEMENT

Sludge management practices would continue with existing handling and disposal methods.

PERFORMANCE

Under the Baseline Condition, increases in wastewater flow would overload all existing STP's during the planning period. The impact on the Susquehanna River is described below:

1. The minimum daily average DO level would be 3.5 mg/l during design low river flow (MA-7-CD-10 flow is 300 cfs at Vestal) and dry wastewater flow conditions.

2. The minimum instantaneous DO would be below 4 mg/l during design storm conditions.

The pollutant mass loads discharged by the STP's, under existing and year 2020 conditions are given in Table VIII-2.

CONSTRUCTION SCHEDULE

With two exceptions, the baseline projects are assumed to be online prior to 1977 and, therefore, no capital costs for these projects have been presented. For implementation by 1977, some degree of uncertainty does exist for the upgrading of the Owego Village STP, as much discussion had centered on the design of this facility and would likely delay construction; and for the Nanticoke Valley interceptor, as this project is currently not rated by NYSDEC for funding. The capital costs associated with the Baseline Plan include an 0.8 mgd regional interceptor to Nanticoke Valley ($1.57 million) and the upgrading of the Owego Village STP ($1.02 million) to a 1.0 mgd secondary plant. The cost for upgrading the Owego Village STP was based on using a trickling filter process; however, current NYSDEC plans call for an activated sludge process.

COST ANALYSIS

Costs associated with the Baseline Plan include the capital costs of the construction schedule, plus the O&M costs accruing to the continued operation of the existing wastewater management facilities. Capital and O&M costs and their present worth are given for the planning period in Table VIII-3. In addition, the present worth of the cost of replacement of the existing facilities is shown. The replacement costs were estimated using an annual sinking fund approach. The present worth costs for each service area and plan components are presented in Table VIII-4. The present worth costs for this plan are: Capital--$2.6 million, O&M--$19.5 million, and Replacement--$9.8 million, for a total cost of $31.9 million.

TABLE VIII-2

MUNICIPAL EFFLUENT CHARACTERISTICS

Existing and Baseline Condition (Year 2020)

STP	Flow (MGD)	SS (lb/day)	BOD[1] (lb/day)	NOD[2] (lb/day)	Total N (lb/day)	Total P (lb/day)
Binghamton-Johnson City						
A) Existing	18.3	4600	3050/3650	5200/8700	2800	700
B) Year 2020	25.3	10000	6350/11200	9400/12800	3480	870
Chenango Valley						
A) Existing		Periodic septic system overflows and malfunctions				
B) Year 2020		Periodic septic system overflows and malfunctions				
Endicott[3]						
A) Existing	4.3	870	870	3060/4600	1010	313
B) Year 2020	9.2	1500	1520	5500/9100	2260	600
Vestal						
A) Existing	1.0	350	1200	1600/1600	350	71
B) Year 2020		To be abandoned and flows sent to Endicott				
East Owego						
A) Existing	0.37	50	30	100/650	175	45
B) Year 2020	2.8	435	435	1450/3020	640	150
West Owego						
A) Existing	0.2	130	90	215/360	78	23
B) Year 2020	0.7	180	180	100/1680	300	80
Owego Village						
A) Existing	0.9	180	300	450/450	97	32
B) Year 2020	0.97	125	125	415/700	150	40

[1,2] summer loadings/winter loadings
single # – year round loadings
[3] assumes solution of present problems and achieves normal trickling filter performance i.e., 85% BOD + SS removal

TABLE VIII-3

PLAN 1

CAPITAL COSTS (MILLION $) AND OPERATION AND MAINTENANCE COSTS ($$10^3$/Yr)

Year	Binghamton Johnson Capital	Binghamton Johnson O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	E. Owego Capital	E. Owego O&M	W. Owego Capital	W. Owego O&M	Owego Village Capital	Owego Village O&M
1977		721			1.57	310		140		40	1.02	70
1983												
1985												
1989												
1990												
1992												
1995												
2000												
2010												
2018												
2027		721				310		140		40		70
Present Worth Million $		11.1			1.57	4.7		2.1		0.6	1.02	1.0
Present Worth of Replacement Million $	5.0				3.5		0.7		0.2		0.4	
Total Present Worth Million $	16.1				9.8		2.8		0.8		2.4	

TABLE VIII-4

PLAN 1 (BASELINE) COSTS
PRESENT WORTH (MILLION $'s)

	Binghamton-Johnson City	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment						
Captial	-	-	-	-	1.0	1.0
O & M	7.3	4.0	1.6	0.4	0.7	14.0
Replacement	4.7	3.3	0.6	0.1	0.3	9.0
TOTAL	12.0	7.3	2.2	0.5	2.0	24.0
Interceptors						
Capital	-	1.6	-	-	-	1.6
O & M	-	-	-	-	-	-
Replacement	-	-	-	-	-	-
TOTAL	-	1.6	-	-	-	1.6
Storm Overflows						
NONE						
Infiltration Control						
NONE						
Sludge						
Capital	-	-	-	-	-	-
O & M	3.8	0.7	0.5	0.2	0.3	5.5
Replacement	0.3	0.2	0.1	0.1	0.1	0.8
TOTAL	4.1	0.9	0.6	0.3	0.4	6.3
Totals						
Capital	-	1.6	-	-	1.0	2.6
O & M	11.1	4.7	2.1	0.6	1.0	19.5
Replacement	5.0	3.5	0.7	0.2	0.4	9.8
TOTAL PLAN COST	16.1	9.8	2.8	0.8	2.4	31.9

Based on 50 year economic life @ 6 1/8% and ENR = 2248.

PLAN 2A

The intent of this plan is to maintain minimum daily average and minimum instantaneous DO levels above 4 mg/l in the Susquehanna River during both low flow and storm overflow conditions, respectivrely. Plans 2A, 2B, and 2C are similar except for the degree of regionalization.

MUNICIPAL WASTEWATER MANAGEMENT

Regionalization of STP's

The Broome County plants would be the existing Binghamton-Johnson City and Endicott STP's. The wastewater generated from the entire Chenango Valley service area would be treated at the Binghamton-Johnson City STP. Tioga County plants would be the existing Town of Owego STP's No. 1 (West Owego) and No. 2 (East Owego) and the Village of Owego STP.

Service areas and interceptors included in this plan are shown in Plate 4. The only additional regional interceptor to those included in the Baseline Plan connects the Chenango Valley service area to the north end of the City of Binghamton's wastewater collection system.

Treatment Levels and Processes

Secondary treatment would be applied at all STP's. The existing activated sludge processes would be used at the Binghamton-Johnson City STP, and at the Town of Owego STP No. 2. The existing trickling filter processes would be used at the Endicott STP and the Town of Owego STP No. 1. The Village of Owego STP was assumed to be upgraded to secondary treatment using a trickling filter process (recently changed to a proposal for an activated sludge process).

Infiltration Control Level in the City of Binghamton

For the level of treatment (secondary) and the regionalization scheme assumed in this plan, an infiltration reduction of 1 mgd was found to be economically justifiable.

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BINGHAMTON WASTEWATER MANAGEMENT STUDY. DESIGN AND COST APPENDI--ETC(U)
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Nonstructural Measures for Flow Reduction

In this plan, the Village of Endicott and the Town of Owego would institute metered user rates. The increased price of water to the consumer should result in a reduction of water consumption, and, therefore, wastewater flows. An educational program to encourage the use of water saving devices could result in an estimated 20 percent reduction in the projected incremental increase in per capita flows. Although such a program could be undertaken, no formal educational activities have been specified in this plan.

STORM OVERFLOW MANAGEMENT

Microscreening units followed by chlorination would be provided near five overflow sites in the City of Binghamton (see Figure VIII-1). This system would maintain a minimum instantaneous DO about 4 mg/l during most storm conditions. A detailed description of the overflow treatment system is given in Chapter VII. An estimate was also made of the costs of inflow control and microscreening treatment for the Village of Owego's combined sewer system.

SLUDGE MANAGEMENT

Several sludge management techniques have been analyzed in Chapter VI. The land application of liquid sludge is recommended, with landfill of dewatered sludge provided as a backup.

PERFORMANCE

Impact on Susquehanna River dissolved oxygen levels:

1. The minimum daily average dissolved oxygen level would be between 4 and 5 mg/l during low river flow and dry wastewater flow conditions by the year 2020.

2. The minimum instantaneous dissolved oxygen level would be between 4 and 5 mg/l during design storm conditions.

The pollution mass loads discharged to the Susquehanna River by the STP's proposed in this alternative are summarized in Table VIII-5.

CONSTRUCTION SCHEDULE

Expansion of the existing wastewater facilities would be required during the planning period to handle the increasing flow expected (see Figures VIII-2 and VIII-3).

At the Binghamton-Johnson City STP, the secondary treatment units would be expanded by the year 1977 by adding two aerators and clarifier units to increase the present 18.3 mgd capacity to 26.5 mgd and by increasing the raw wastewater pumping capacity from 18.3 mgd to 29.0 mgd. A primary tank would be added by the year 1997 to increase the capacity from 25.0 mgd to 29.0 mgd (30' x 87' x 12' depth). A third aerator and clarifier unit, increasing the capacity from 26.5 mgd to 30.5 mgd, would provide the necessary expansion of the secondary units by the year 2025 (see Figure VIII-4). The Chenango service area wastewater would be transmitted to the City of Binghamton's wastewater collection system through a 16" force main with a design flow of 2.2 mgd. A 1.38 million gallon holding tank would be used to equalize the wastewater flow and reduce the pump station and the force main costs. This interceptor would be built by the year 1977.

Storm overflow control facilities are assumed to be constructed at the City of Binghamton by the year 1977. Five microscreening facilities having a total treatment capacity of 39.5 mgd would be located as shown in Figure VIII-1.

At the Endicott STP, the secondary treatment units will need expansion from 7.7 mgd to 9.2 mgd capacity by the year 1983. The trickling filters, secondary clarifitions, digesters, chlorination tank, raw wastewater, and sludge pumping capacities will be expanded (see Figure VIII-5).

The Nanticoke service area wastewater would be transmitted to the Endicott STP through an 18-inch diameter sewer that has a design average flow of 0.8 mgd and a peak flow of 2.3 mgd. The sewer length would be 25,000 feet. This interceptor would be built by the year 1977.

At the Town of Owego STP No. 2, the plant treatment capacity will be upgraded from the present 2.0 mgd to 3.0 mgd by the year 1992. The additional units include preliminary treatment; a primary settling tank with a 1,250 square foot capacity; a 31,000 cubic foot aeration tank and air system; a sludge thickener for 4 mgd; a digester having a 45' diameter with a 20' depth; a chlorination tank 25' by 18' and 8' deep; and two final clarifiers having a 3 mgd capacity and each is 55' by 12' and 8.5' deep (see Figure VIII-6).

At the Town of Owego STP No. 1, the plant capacity will be expanded by the year 2000 from the present 0.5 mgd to 0.7 mgd. The added facilities will include a preliminary treatment capacity, a final settling tank, and a chlorination tank (see Figure VIII-7).

At the Village of Owego, the treatment facilities will be upgraded to provide secondary treatment by the year 1977. The capacity of the upgraded facility will be 1 mgd. The facilities added will include a trickling filter with a surface area of 4,370 square feet, 75' diameter and 5'6" deep; a secondary pump station designed to handle a maximum flow of 3.0 mgd, two final settling tanks each 14' by 68' and 7' deep; chlorine tank, volume 3,800 cubic feet; two sludge holding tanks with mixers, each 19' by 22' and 9' deep; and a vacuum filter with an area of 100 square feet (see Figure VIII-8).

At the Village of Owego, a microscreening treatment unit with a capacity of 3 mgd will be built by the year 1977 to provide treatment of storm overflows. Also, steps will be taken to control direct stream inflow into the existing system.

A summary of the construction schedule of wastewater treatment facilities needed at each service area is given in Table VIII-6.

COST ANALYSIS

The capital costs breakdown for the expansion and/or upgrading of the existing treatment facilities at Binghamton-Johnson City, the Village of Endicott, the Town of Owego STP No. 1, Town of Owego STP No. 2, and the Village of Owego are shown in Tables VIII-7 through VIII-11.

The capital costs breakdown for the Chenango interceptor and the Nanticoke interceptor are shown in Tables VIII-12 and VIII-13.

The operation and maintenance costs of the wastewater treatment plants for each service area during the planning period are given in Table VIII-14.

The expenditures for each service area are summarized in Table VIII-15. Also shown in the table are the years these expenditures will be incurred. These correspond to the construction schedule described in the previous section. The replacement costs were estimated using an equivalent annual sinking fund approach.

The present worth of all expenditures during the 50-year planning period is given in Table VIII-16 for each service area. The present worth costs for this plan are: capital -- \$13.5 million, O&M -- \$22.1 million, and replacement -- \$11.6 million, for a total cost of \$47.2 million.

CITY OF BINGHAMTON
LOCATION OF PLANNED OVERFLOW CONTROL FACILITIES

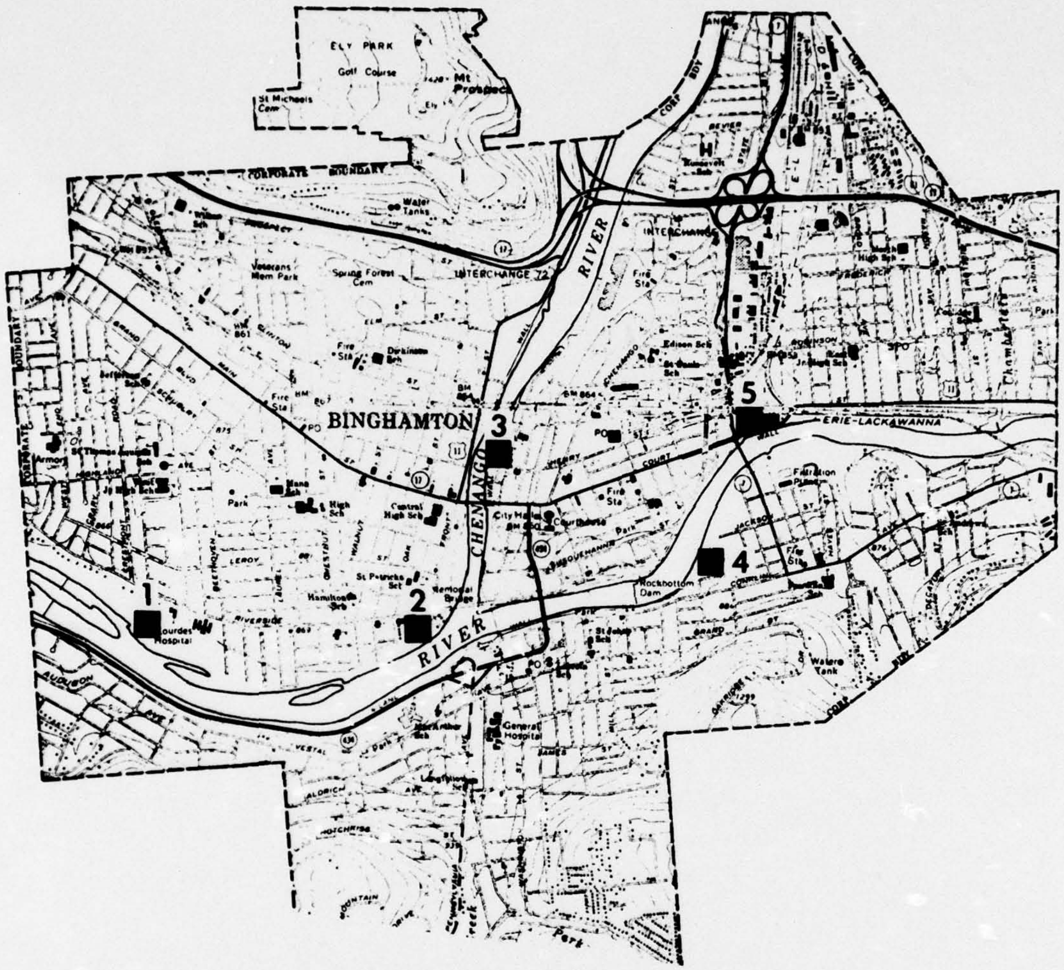


FIGURE VIII-1

TABLE VIII-5

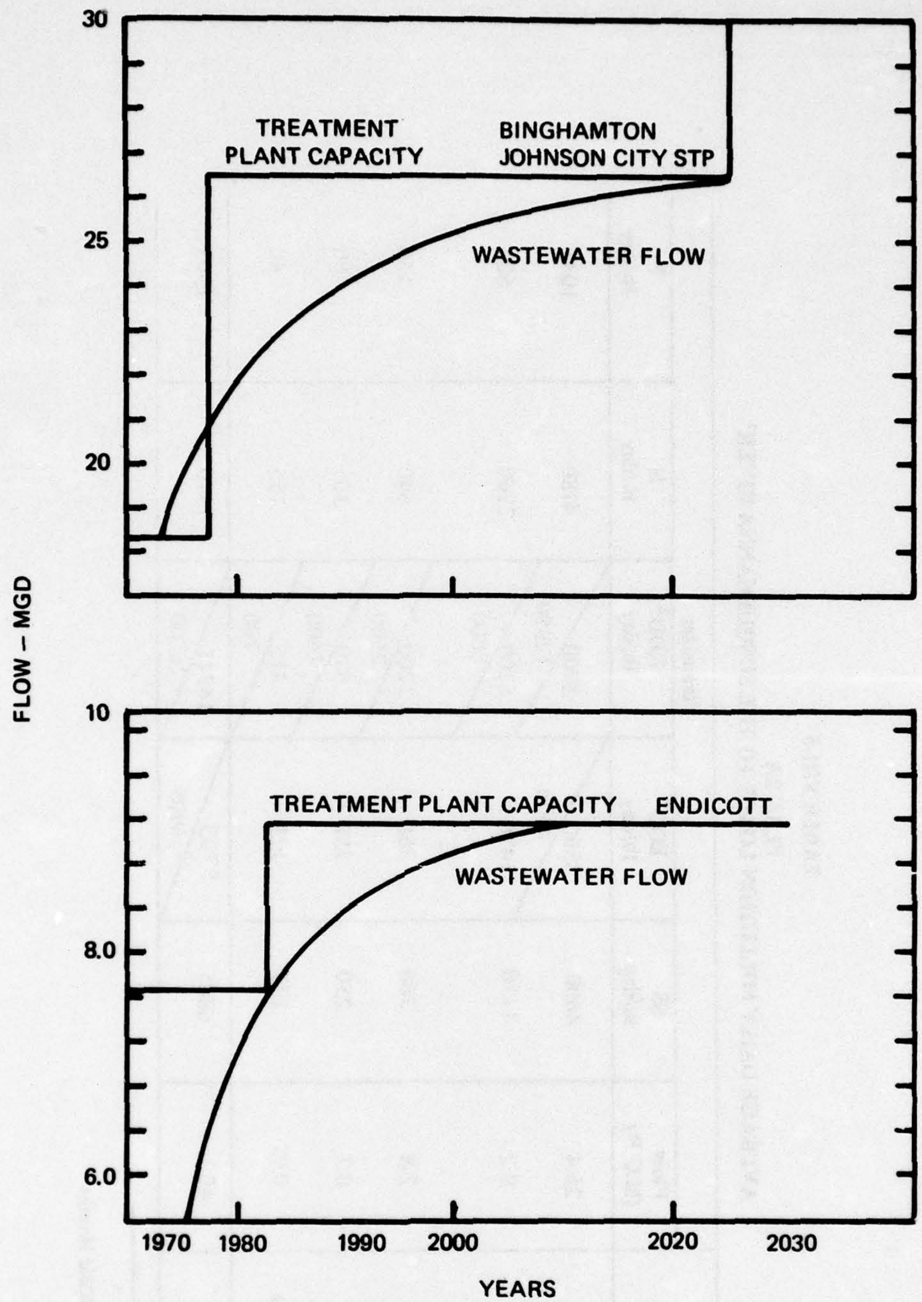
PLAN 2A

AVERAGE DAILY POLLUTION LOADS TO THE SUSQUEHANNA RIVER[1]

Source	Parameter					
	Flow (M.G.D)	SS lb/day	BOD[2] lb/day	NOD[2] lb/day	N lb/day	P lb/day
Binghamton-Johnson	26.4	4600	3500 / 4700	8900 / 12900	4180	1035
Endicott	9.2	1470	1470	5200 / 8800	2260	600
Chenango	-	-	-	-		
Owego #2	2.8	360	360	1200 / 2500	640	150
Owego #1	0.7	250	250	830 / 1400	305	80
Owego Village	0.97	125	125	415 / 700	155	40
Total	40.1	6805	5705 / 6905	16345 / 26300	7540	1905

[1] Year 2020

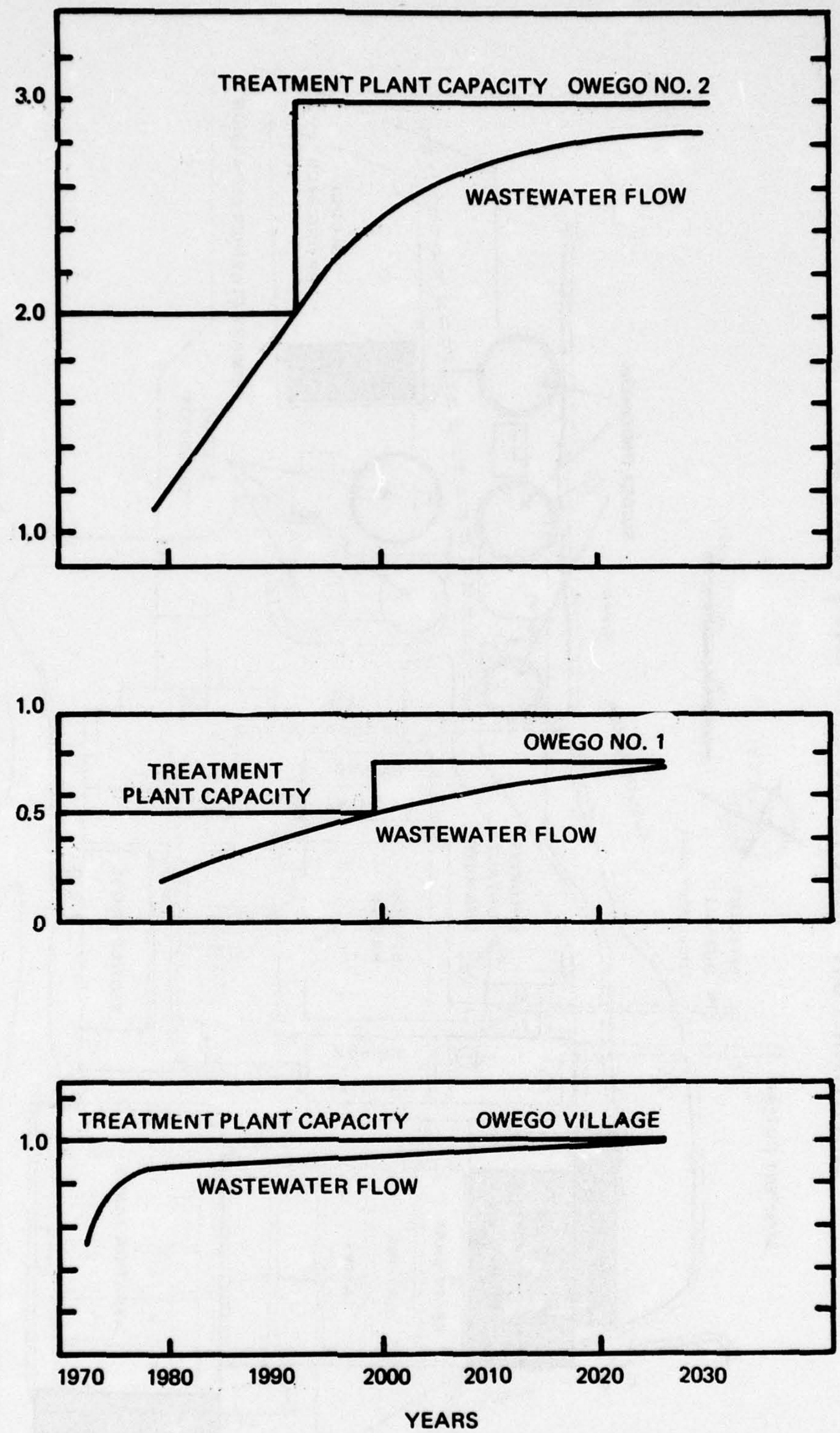
[2] Warm Months/Cold Months



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – BROOME COUNTY

FIGURE VIII-2

PLAN 2A



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – TIOGA COUNTY

FIGURE VIII-3

PLAN 2A, 2B, 2C, 3A, 3B, and 3C

BINGHAMTON - JOHNSON CITY STP, PLAN 2A

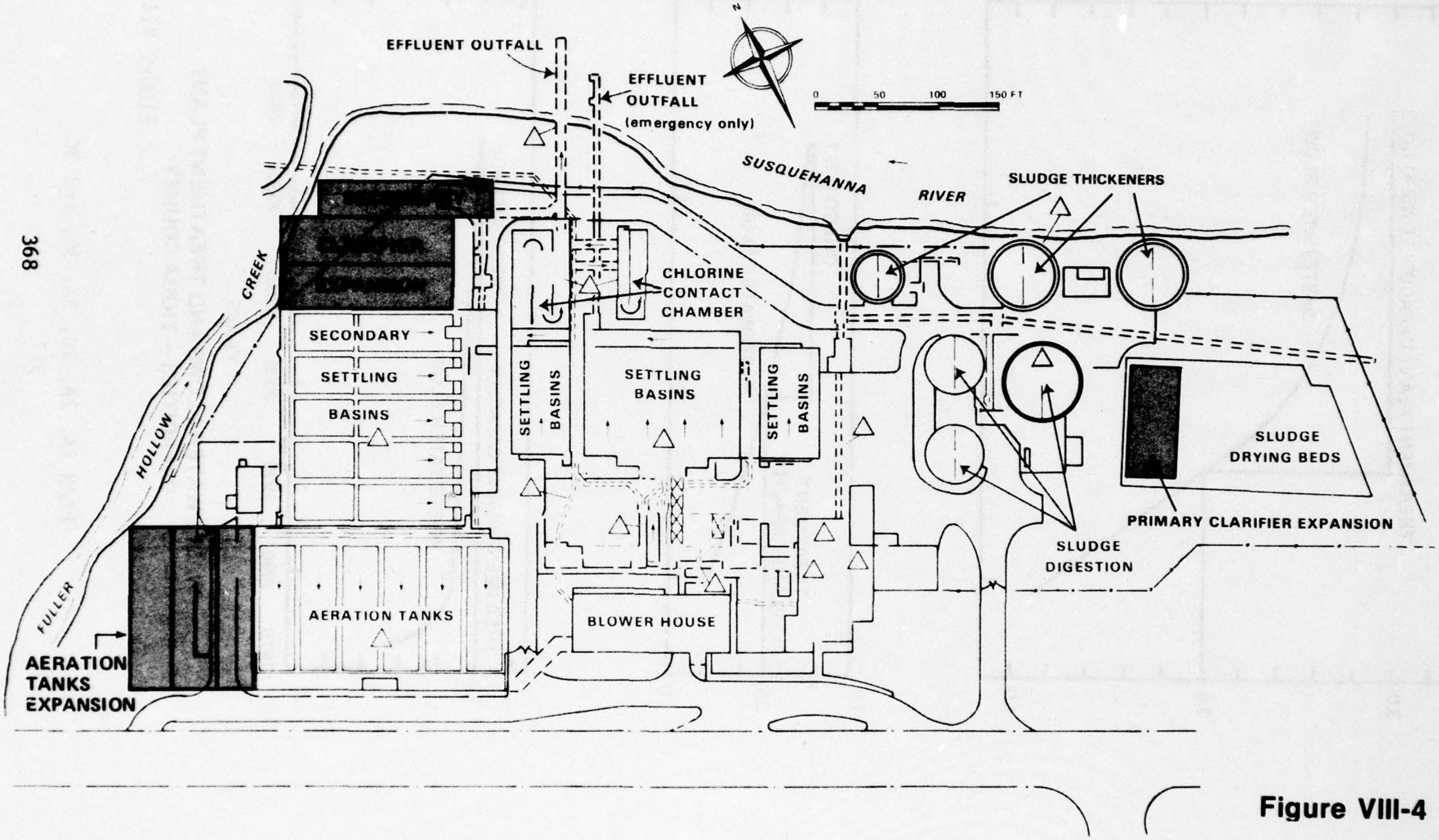


Figure VIII-4

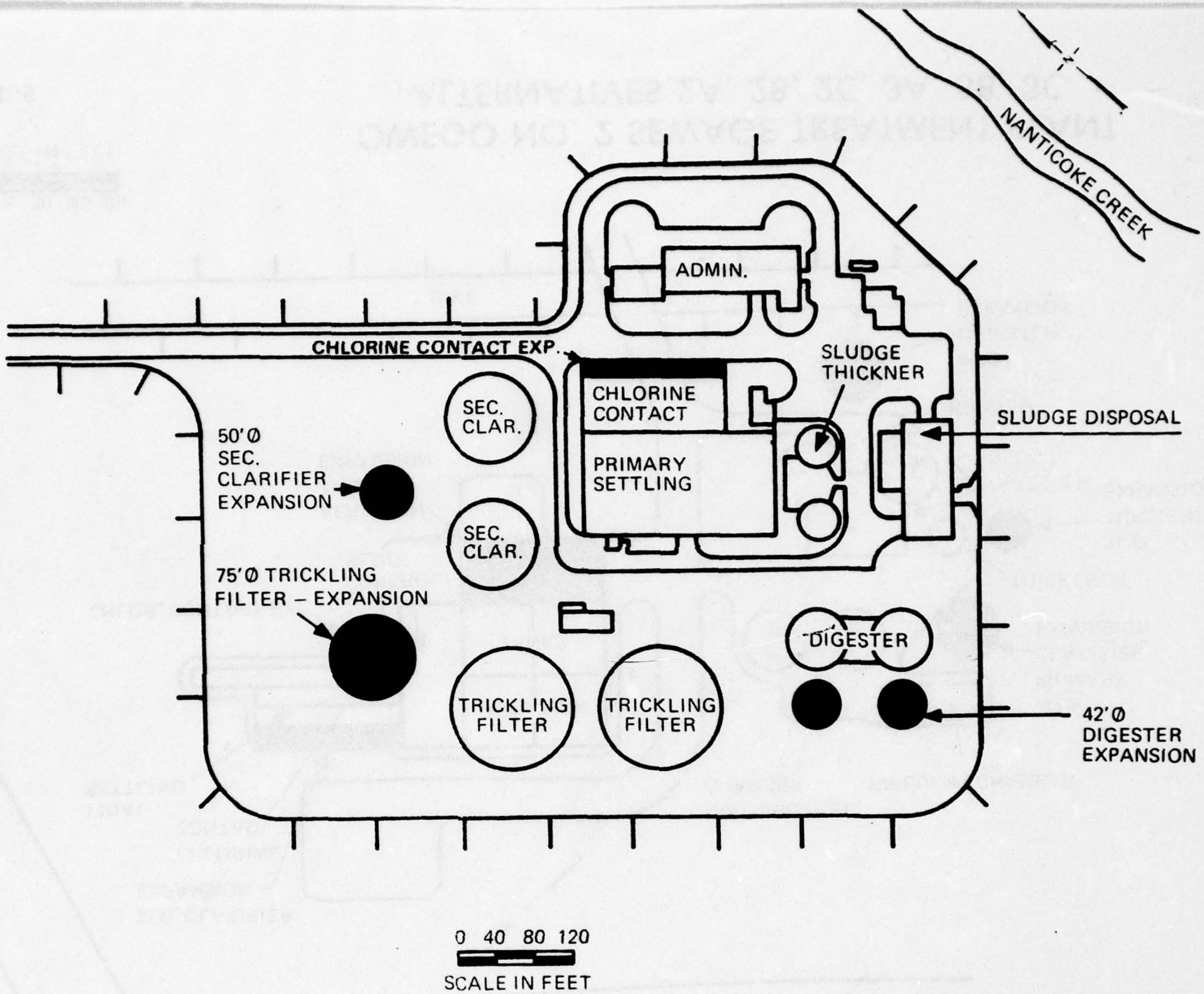
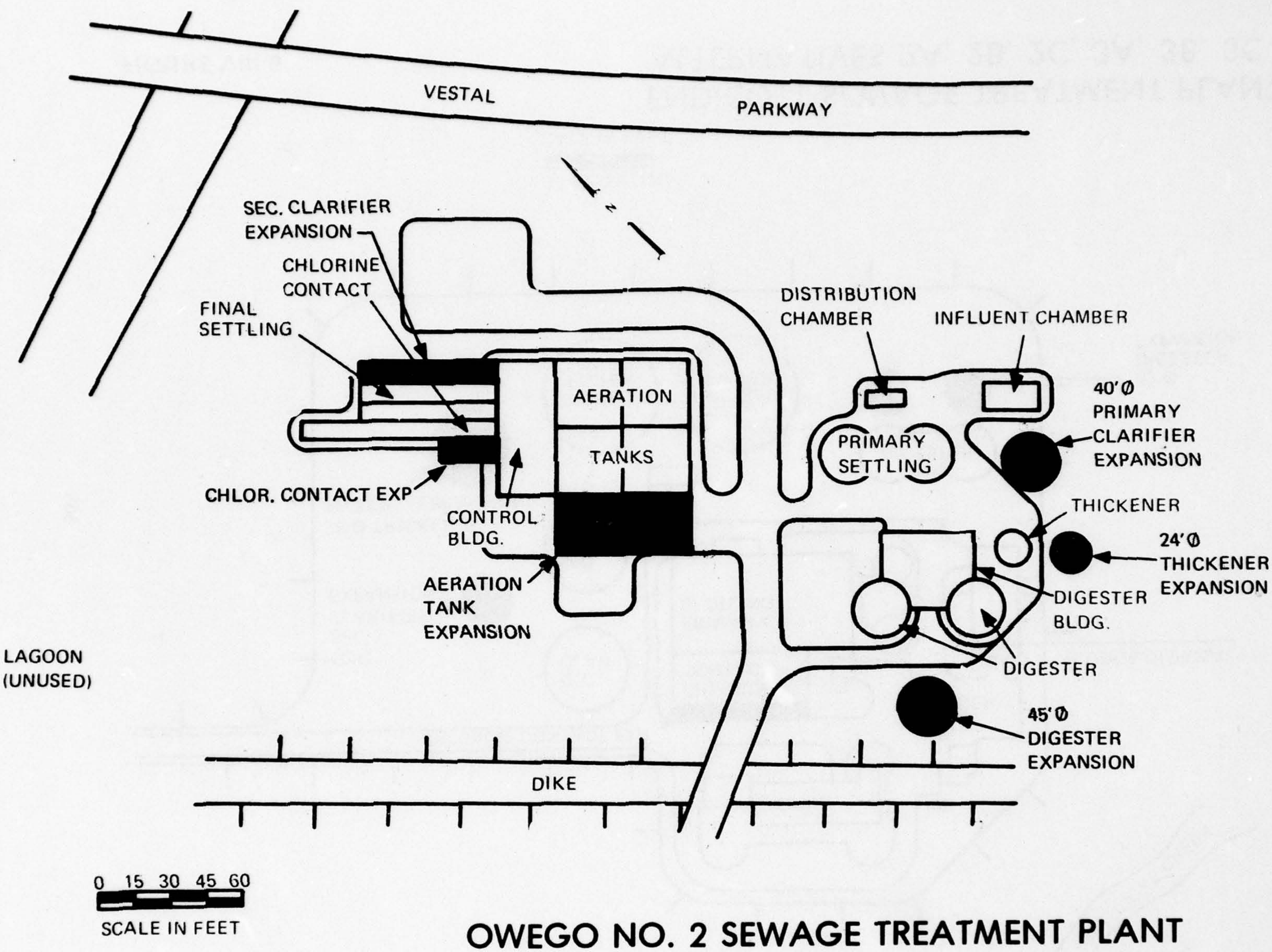


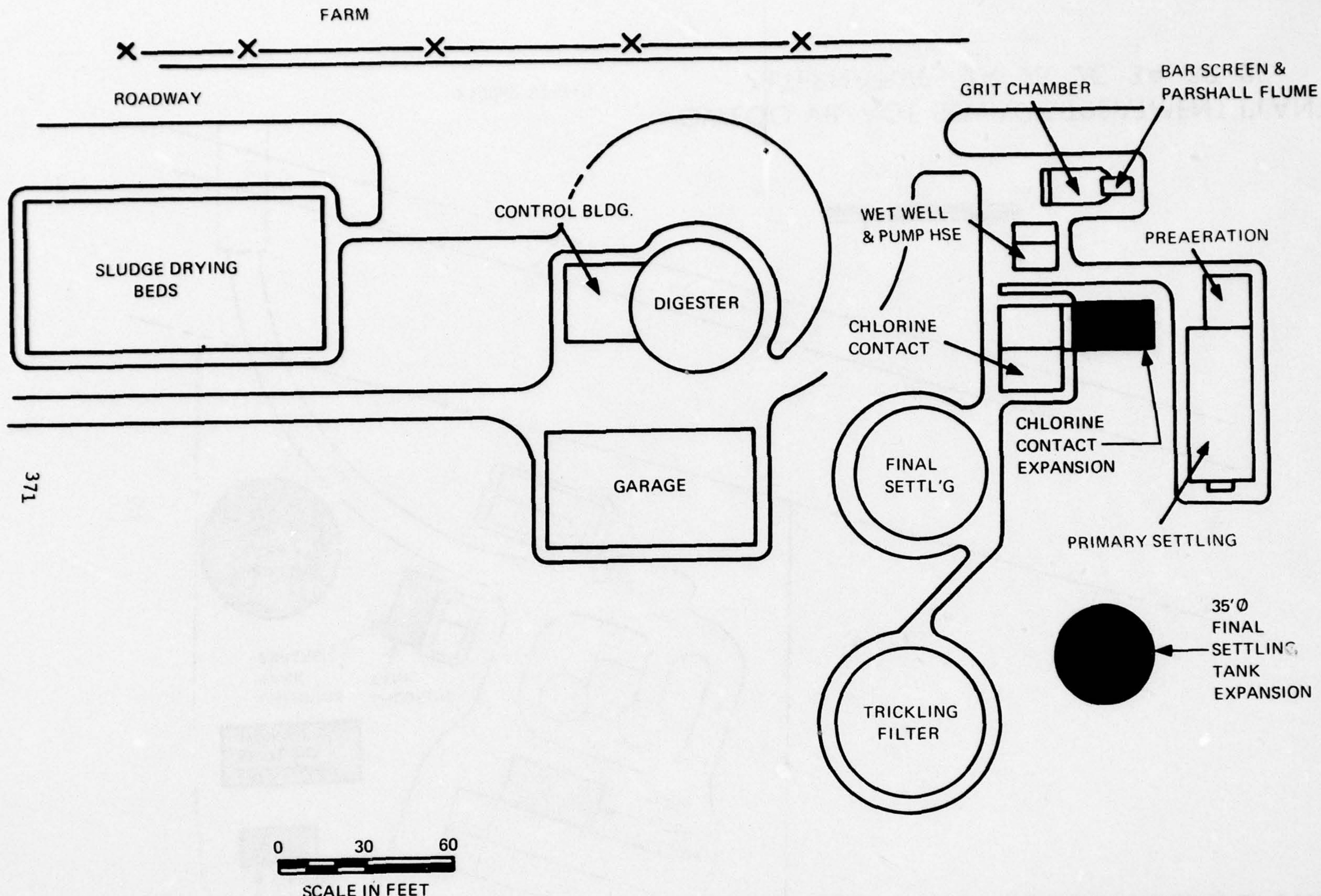
FIGURE VIII-5

ENDICOTT SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C



OWEGO NO. 2 SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-6



OWEGO NO. 1 SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-7

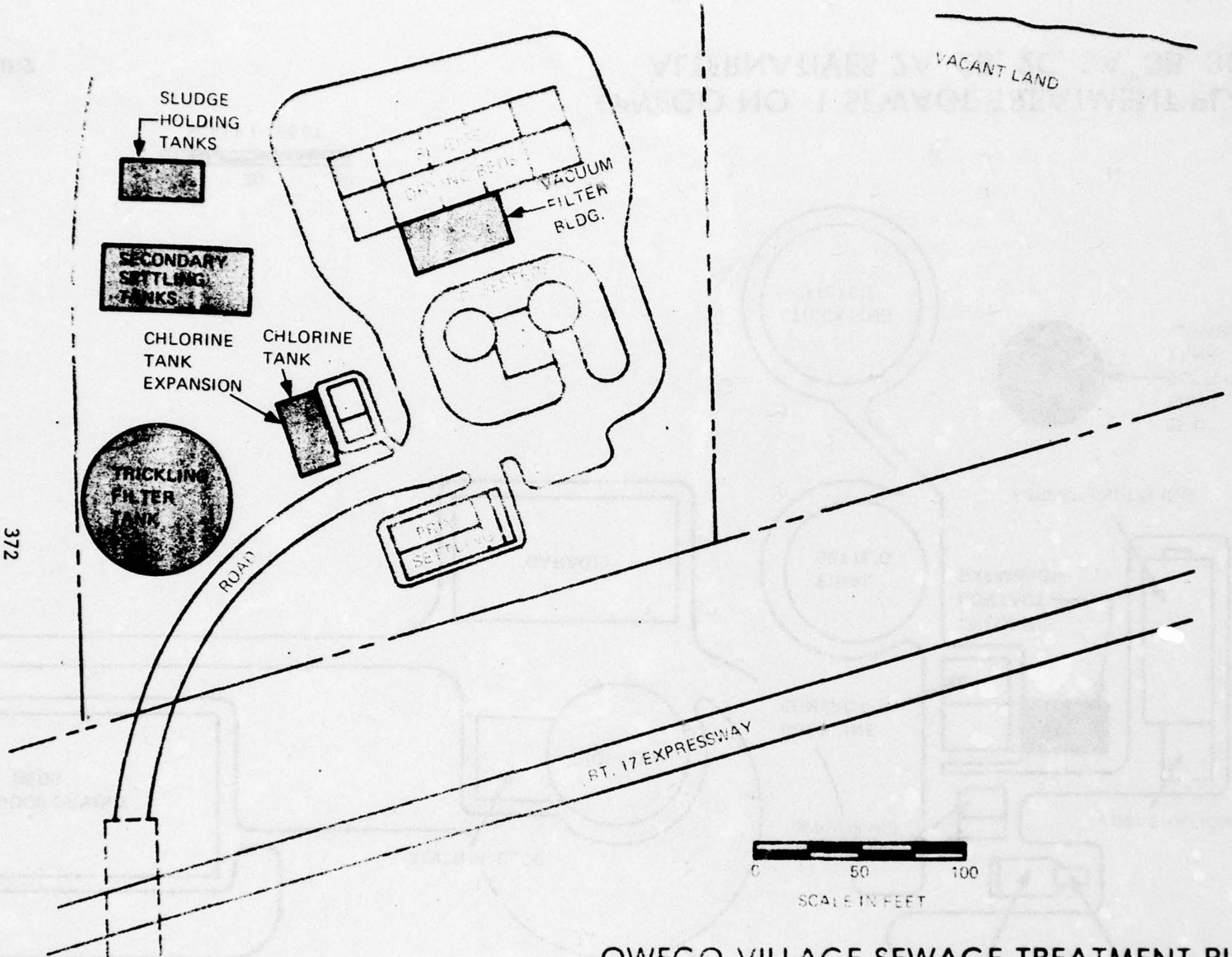


FIGURE VIII-8

OWEGO VILLAGE SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

TABLE VIII-6

PLAN 2A
CONSTRUCTION SCHEDULE

	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth [1] ($ Million)	Description
Binghamton-Johnson City	2.2	1977	2.50		Chenango Interceptor
	1.0	1977	0.21		Infiltration Control
	39.5	1977	3.58		Storm Overflow
	29.0	1977	0.05		Raw Wastewater Pumping
	26.5	1977	1.42		Aerator and Clarifier (2 sets)
	29.0	1997	0.39		Primary Tank
	30.5	2025	0.71		Aeration and Clarifier (1 set)
				8.0	
Endicott	0.8	1977	1.57		Nanticoke Valley Interceptor
	9.2	1983	1.95		Additional Secondary Treatment Capacity
				2.9	
East Owego (#2)	3.0	1992	1.26		General Expansion
				0.5	
West Owego (#1)	0.7	2000	0.47		General Expansion
				0.1	
Owego Village	1.0	1977	1.02		New Secondary treatment capability
		1977	0.58		Inflow Control
	3.0	1977	0.42		Micro-screening
				2.0	
Total Present Worth				13.5	

[1] 50 years @ 6-1/8%

TABLE VIII-7

CAPITAL COST BREAKDOWN FOR AN AERATION TANK AND SECONDARY CLARIFIER SET AT THE BINGHAMTON–JOHNSON CITY STP*

1)	Clarifier	$190,000
2)	Aeration Tank (30′ X 100′)	225,000
3)	Yard Piping & Renovation Instrumentation	80,000
4)	Additional Pumping and Valves	50,000
	Sub Total	545,000
	Eng. & Cont. @ 30%	164,000
	Total	$709,000

*July 1975 ENR 2248

TABLE VIII-8

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE SECONDARY TREATMENT FACILITIES AT ENDICOTT STP FROM 7.7 TO 9.2 MGD CAPACITY*

1)	Preliminary Treatment	44,000
2)	Raw Wastes Pumps	74,000
3)	Trickling Filter (75'φ - 6'deep)	105,000
4)	Secondary Clarifier (50'φ X B'- 6 SWD)	158,000
5)	Chlorine Tank & Feed	51,000
6)	Digester Incl Bld Co. (42'φ - 30"high)	440,000
7)	Sludge Pumping	84,000
8)	Piles (for all structures)	345,000
9)	Misc. Electricity, Heating and Ventilation	148,000
10)	Yard Piping	54,000
	Subtotal	1,503,000
	Engineering & Contingencies @ 30%	451,000
	Total	1,954,000

*July 1975 ENR 2248.

TABLE VIII-9

CAPITAL COST BREAKDOWN FOR EXPANSION OF THE TOWN OF OWEGO STP NO. 2 CAPACITY FROM 2.0 MGD TO 3 MGD*

1)	Prelimination Treatment & R.W. Pumps, wet well	137,000
2)	Preliminary Settling Tanks 1 tank – 1250 SF	63,000
3)	Aeration Tank & Air System 31,000 CF	116,000
4)	Pumps – Recirculating Piping, Structure Upgrade	80,000
5)	Sludge Thickener (for 4 MGD 24′ Diameter)	42,000
6)	Digester 45′ ϕ X 20′ Diameter	220,000
7)	Chlorine Tank & Feed System	58,000
8)	Sludge Pumping	47,000
9)	Final Clarifiers 55′ X 12′ X 8.5′ depth	74,000
10)	Misc. – Electricity, Heating and Ventilation Sitework	132,000
	Subtotal	969,000
	30% Engr & Cont	291,000
	Total	1,260,000

*July 1975 ENR 2248.

TABLE VIII-10

CAPITAL COST BREAKDOWN FOR EXPANSION OF THE TOWN OF OWEGO STP NO. 1 CAPACITY FROM 0.5 MGD TO 0.7 MGD*

1)	Preliminary Treatment	26,000
2)	R.W. Pumps, Internal Piping	90,000
3)	Final Settling Tank (35′ϕ, Dept = 8′)	74,000
4)	Chlorination Tank (16′ X 40′ X 6′)	47,000
5)	Misc. Electrical Instrumentation Heating and Vent.	58,000
6)	Yard Piping, Effluent Piping	68,000
	Subtotal	363,000
	30% Engr. & Cont.	109,000
	Total	472,000

*July 1975 ENR 2248.

TABLE VIII-11

CAPITAL COST BREAKDOWN FOR UPGRADING THE VILLAGE OF OWEGO STP SECONDARY TREATMENT CAPACITY TO 1 MGD*

1)	Secondary Pump Station (Max Flow = 3.0)	200,000
2)	Trickling Filter A = 4,370 SF 75'ϕ X 5'-6" Stone Depth	87,000
3)	Final Settling TKS A = 1,904 SF 2 - 14' X 68' X 7' SWD	78,000
4)	Chlorine Tanks Vol = 3,800 16' X 40' X 6' SWD	25,000
5)	Sludge Holding Tanks 2 - 19' X 22' X 9' SWD With Mixers	60,000
6)	Vacuum Filter Area = 100 SF including Building 8'ϕ X 4' Face	160,000
7)	Preliminary Treatment – Screens, Grit removal Conninutor, etc.	65,000
8)	Site Work – Added Fill, Yard Piping, Grading, etc.	110,000
	Subtotal	785,000
	30% Engr. & Cont.	235,000
	Total	1,020,000

*July 1975 ENR 2248.

TABLE VIII-12

CAPITAL COST BREAKDOWN
FOR THE CHENANGO INTERCEPTOR
(AVERAGE DESIGN FLOW = 2.2 M.G.D.
MAXIMUM DESIGN FLOW = 3.3 M.G.D. FOR 10 HOURS)*

1) Pump Station Q = 2.2 M.G.D. (Aug.)	265,000
2) Holding Basin with Aerators (125' x 125' = 1.375 MG)	
a) Concrete	515,000
b) Excavation	53,000
c) Aerators	160,000
d) Piling	145,000
e) Piping Instr. Values, etc.	180,000
Subtotal	1,053,000
3) Dewatering, Sheeting, Paving, etc.	53,000
4) Force Main, Q = 2.2 M.G.D. 16" – 13,500' @ $33.70/LF	455,000
5) Highway Crossing, 30" casing – 250' @ $264/LF	66,000
6) Land	32,000
Subtotal	1,924,000
30% Engr. & Contr.	577,000
Total	2,501,000

*July 1975 ENR 2248.

TABLE VIII-13

CAPITAL COST BREAKDOWN FOR THE NANTICOKE INTERCEPTOR (AVERAGE DESIGN FLOW = 0.8 M.G.D.* PEAK DESIGN FLOW = 2.3 M.G.D.)

1)	18" sewer – 25,000' @ $38/LF	957,000
2)	110 manholes @ $1,100	120,000
3)	Misc. items	33,000
4)	Railroad crossing 27" casing 100' @ $270	27,000
5)	Creek Crossing 200/LF @ $350/LF	70,000
	Subtotal	1,207,000
	30% Engr. & Contr.	362,000
	Total	1,569,000

*use 18"φ sewer @ S = 0.0012 min. assume avg. depth = 12-14 feet.

July 1975 ENR 2248.

TABLE VIII-14

PLAN 2A
OPERATION AND MAINTENANCE COSTS

	First Year	Last Year	Description	Annual Cost ($ Million/Yr)	Present Worth[1] ($ Million)
Binghamton-	1977	2026	Secondary	0.78	
Johnson City	1977	2026	Storm Overflow	0.05	
					12.9
Endicott	1977	1982	Secondary	0.31	
	1983	2026	Secondary	0.36	
					5.1
East Owego	1977	1991	Secondary	0.14	
	1992	2026	Secondary	0.19	
					2.4
West Owego	1977	1999	Secondary	0.04	
	2000	2026	Secondary	0.05	
					0.6
Owego Village	1977	2026	Microscreening	0.002	
	1977	2026	Secondary	0.07	
					1.1
				Total Present Worth:	22.1

[1] 50 years @ 6-1/8%.

TABLE VIII-15

PLAN 2A

CAPITAL COSTS (MILLION $) AND OPERATION AND MAINTENANCE COSTS ($$10^3$/Yr)

Year	Binghamton Johnson Capital	Binghamton Johnson O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	E. Owego Capital	E. Owego O&M	W. Owego Capital	W. Owego O&M	Owego Village Capital	Owego Village O&M
1977	7.76	830			1.57	310		140		40	2.0	70
1983					1.95	360						
1985												
1989												
1990												
1992							1.26	190				
1995												
1997	0.39											
2000									.47	50		
2010												
2018												
2025	.71											
2026						360		190		50		70
Present Worth Million $	8.0	12.9			2.9	5.1	0.5	2.4	0.1	0.6	2.0	1.1
Present Worth of Replacement Million $	6.3				3.8		0.8		0.2		0.5	
Total Present Worth Million $	27.2				11.8		3.7		0.9		3.6	

TABLE VIII-16

PLAN 2A PRESENT WORTH[1] (MILLION $'s)

	Binghamton-Johnson City	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment						
Captial	1.6	1.3	0.5	0.1	1.0	4.5
O & M	7.8	4.4	1.9	0.4	0.7	15.2
Replacement	5.1	3.6	0.7	0.1	0.3	9.8
TOTAL	14.5	9.3	3.1	0.6	2.0	29.5
Interceptors						
Captial	2.5	1.6				4.1
O & M	-	-				-
Replacement	-	-				-
TOTAL	2.5	1.6				4.1
Storm Overflows						
Capital	3.6	-	-	-	1.0	4.6
O & M	0.8	-	-	-	0.1	0.9
Replacement	0.9	-	-	-	0.1	1.0
TOTAL	5.3				1.2	6.5
Infiltration Control						
Capital	0.2	-	-	-	-	0.2
O & M	-	-	-	-	-	-
Replacement	-	-	-	-	-	-
TOTAL	0.2					0.2
Sludge						
Capital	0.05	0.01	0.005	0.001	0.002	0.1
O & M	4.3	0.7	0.5	0.2	0.3	6.0
Replacement	0.3	0.2	0.1	0.1	0.1	0.8
TOTAL	4.7	0.9	0.6	0.3	0.4	6.9
Totals						
Capital	8.0	2.9	0.5	0.1	2.0	13.5
O & M	12.9	5.1	2.4	0.6	1.1	22.1
Replacement	6.3	3.8	0.8	0.2	0.5	11.6
Total Plan Cost	27.2	11.8	3.7	0.9	3.6	47.2

[1] Based on a 50 year economic life @ 6 1/8% and ENR = 2248.

PLAN 2B

The intent of this plan is to maintain minimum daily average and minimum instantaneous DO levels above 4 mg/l in the Susquehanna River during both low flow and storm overflow conditions, respectively. Plans 2A, 2B, and 2C are similar except for the degree of regionalization.

MUNICIPAL WASTEWATER MANAGEMENT

Regionalization of STP's

The Broome County plants would be the existing Binghamton -- Johnson City and Endicott STP's and a new STP to serve the entire Chenango Valley service area. Tioga County plants would be the Town of Owego STP's No. 1 (West Owego) and No. 2 (East Owego) and the Village of Owego STP (all existing).

Services and interceptors included in this plan are shown in Plate 5. With the exception of the Chenango Valley sewerage system, these are the same interceptors included in the Baseline Plan.

Treatment Levels and Processes

Secondary treatment would be applied at all STP's. The existing activated sludge processes would be used at the Binghamton-Johnson City STP and at the Town of Owego STP No. 2. A new activated sludge treatment plant would be built in Chenango Valley. The existing trickling filter processes would be used at the Endicott STP and the Town of Owego STP No. 1. The Village of Owego STP was assumed to be upgraded to secondary treatment using a trickling filter process. (recently changed to a proposal for an activated sludge process).

Infiltration Control Level in the City of Binghamton

For the level of treatment (secondary) and the regionalization scheme assumed in this plan, an infiltration reduction of 1 mgd was found to be economically justifiable.

Nonstructural Measures for Flow Reduction

In this plan, the Village of Endicott and the Town of Owego would institute metered user rates. The increased price of water to the consumer should result in a reduction of water consumption, and, therefore, wastewater flows.

An educational program to encourage the use of water saving devices could result in an estimated 20 percent reduction in the projected incremental increase in per capita flows. Although such a program could be undertaken, no formal educational activities have been specified in this plan.

STORM OVERFLOW MANAGEMENT

Microscreening units followed by chlorination would be provided near five overflow sites in the City of Binghamton (see Figure VIII-9). This system would maintain a minimum instantaneous DO about 4 mg/l during most storm conditions. Amore detailed description of the overflow treatment system is given in Chapter VII. An estimate was also made of the costs of inflow control and microscreening treatment for the Village of Owego is combined sewer system.

SLUDGE MANAGEMENT

Several sludge management techniques have been analyzed in Chapter VI. .The land application of liquid sludge was recommended, with landfill of dewatered sludge provided as a backup.

PERFORMANCE

Impact on Susquehanna River dissolved oxygen levels:

1. The minimum daily average dissolved oxygen level would be between 4 and 5 mg/l during low river flow and dry wastewater flow conditions.

2. The minimum instantaneous dissolved oxygen level would be between 4 and 5 mg/l during design storm conditions.

The pollutant mass loads discharged to the Susquehanna River by the STP's proposed in this plan are summarized in Table VIII-17.

CONSTRUCTION SCHEDULE

The Chenango Valley STP would be built by 1977 to conform with the Federal requirements for secondary treatment (see Figure VIII-10). The plant would have an initial capacity of 1.7 mgd, to be expanded to 2.2 mgd by 1985.

Expansion of the wastewater facilities would be required during the planning period at all STP's to handle the increasing flow expected (see Figures VIII-11 and VIII-12).

At the Binghamton-Johnson City STP, the raw wastewaters pumping capacity will be increased from 18.3 mgd to 29 mgd by the year 1977. Also by this year, one aerator and clarifier unit will be added to increase the capacity from 18.3 mgd to 22.2 mgd, with another aeration and clarifier unit to be added by the year 1991 to increase the capacity from 22.2 mgd to 25.4 mgd (see Figure VIII-13).

Storm overflow control facilities are assumed to be constructed at the City of Binghamton by the year 1977. Five microscreening facilities having a total treatment capacity of 39.5 mgd will be located as shown in Figure VIII-9).

At the Endicott STP, the secondary treatment units will need expansion from 7.7 mgd to 9.2 mgd capacity by the year 1983. The trickling filters, secondary clarifitions, digesters, chlorination tank, raw wastewater and sludge pumping capacities will be expanded (see Figure VIII-14).

The Nanticoke service area wastewater will be transmitted to the Endicott STP through an 18 inch diameter sewer that has a design average flow of 0.8 mgd and a peak flow of 2.3 mgd. The sewer length would be 25,000 feet. This interceptor would be built by the year 1977.

At the Town of Owego STP No. 2, the plant treatment capacity will be upgraded from the present 2.0 mgd to 3.0 mgd by the year 1992. The additional units include preliminary treatment; a primary settling tank with a 1,250 square foot capacity; a 31,000 cubic foot aeration tank and air system; a sludge thickener for 4 mgd; a digester having a 45' diameter with a 20' depth; a chlorine tank, 25' by 18' and 8' deep; and two final clarifiers having a 3 mgd capacity, each being 55' by 12' and 8.5' deep (see Figure VIII-15).

At the Town of Owego STP No. 1, the plant capacity will be expanded by the year 2000 from the present 0.5 mgd to 0.7 mgd. The added facilities will include a preliminary treatment capacity, a final settling tank, and a chlorination tank (see Figure VIII-16).

At the Village of Owego, the treatment facilities will be upgraded to provide secondary treatment by the year 1977. The capacity of the upgraded facility will be 1 mgd. The facilities added will include a trickling filter with a surface area of 4,370 square feet, 75' diameter and 5'6" depth; a secondary pump station designed to handle a maximum flow of 3.0 mgd; two final settling tanks, each 14' by 68' and 7' deep; chlorine tanks volume 3,800 cubic feet; two sludge holding tanks with mixers each 19' by 22' and 9' deep; and a vacuum filter with an area of 100 square feet (see Figure VIII-17).

At the Village of Owego, a microscreening treatment unit with a capacity of 3 mgd will be built by the year 1977 to provide treatment of storm overflows. Also, steps will be taken to control direct stream inflow into the existing system.

A summary of the construction schedule of wastewater treatment facilities needed at each service area is given in Table VIII-18.

COST ANALYSIS

The capital costs breakdown for the expansion and/or upgrading of the existing treatment facilities at Binghamton -- Johnson City, the Village of Endicott, the Town of Owego STP #1, Town of Owego STP #2, and the Village of Owego are shown in Tables VIII-19 through VIII-23. The capital costs breakdown for the new Chenango Valley STP and the Nanticoke interceptor are shown in Tables VIII-24 and VIII-25, respectively.

The O&M costs for each treatment plant during the planning period are given in Table VIII-26. The capital, O&M, and replacement costs for each service area are summarized in Table VIII-27. Also shown in the Table are the years these expenditures would be incurred, which correspond to the construction schedule described in the previous section.

The present worth of all the expenditures during the 50 year planning period is given in Table VIII-28 for each service

area. The present worth costs for this plan are: Capital -- $12.6 million, O&M -- $23.2 million, and replacement -- $11.9 million, for a total cost of $47.7 million.

CITY OF BINGHAMTON
LOCATION OF PLANNED OVERFLOW CONTROL FACILITIES

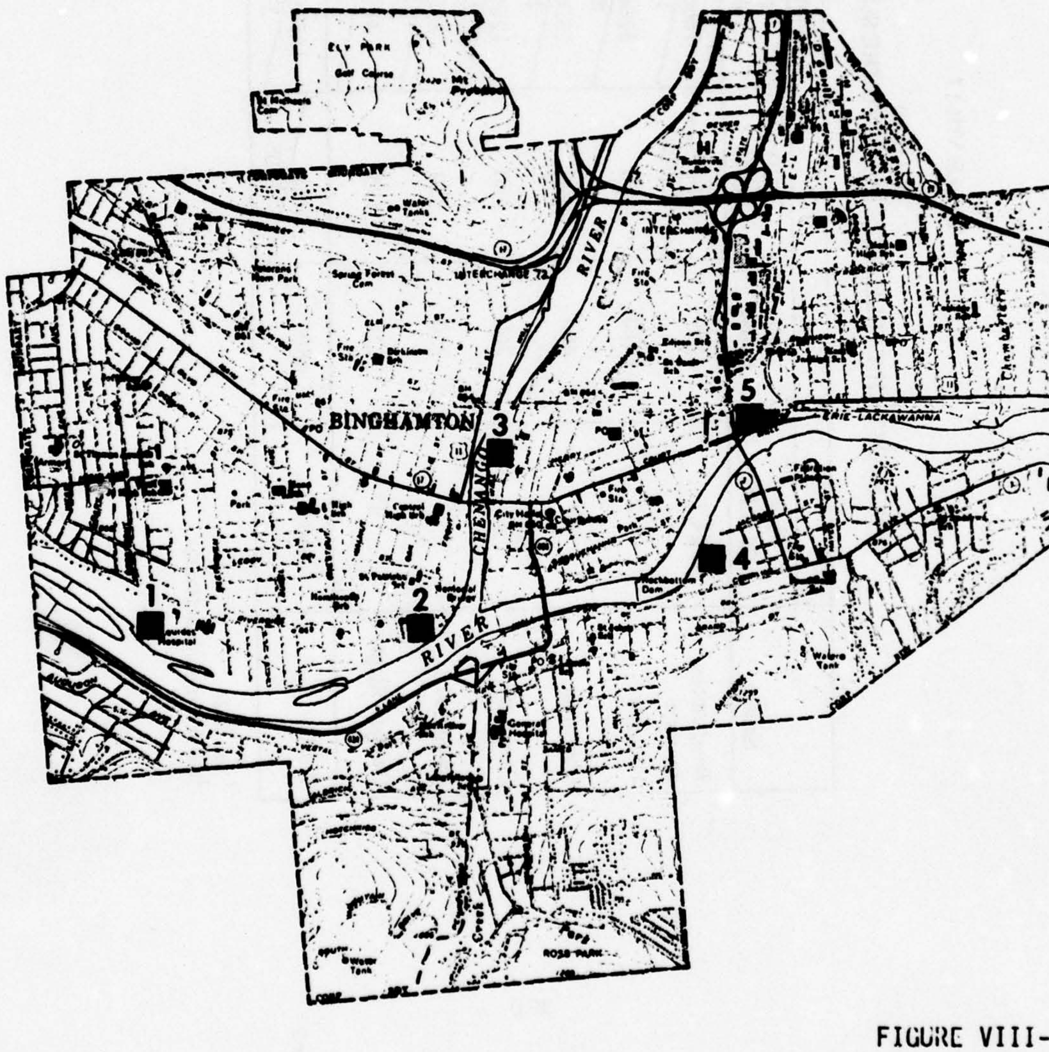


FIGURE VIII-9

TABLE VIII-17

PLAN 2B
AVERAGE DAILY POLLUTION LOADS TO THE SUSQUEHANNA RIVER[1]

Source	Parameter					
	Flow (M.G.D)	SS lb/day	BOD[2] lb/day	NOD[2] lb/day	N lb/day	P lb/day
Binghamton-Johnson	24.2	4200	3100/4700	7600/12900	3470	870
Endicott	9.2	1470	1470	5200/8800	2260	600
Chenango	2.2	400	400	1300/2800	710	165
Owego #2	2.8	360	360	1200/2500	640	150
Owego #1	.7	250	250	830/1400	305	80
Owego Village	.97	125	125	415/700	155	40
Total	40.1	6805	5705/7305	16545/29100	7540	1905

[1]Year 2020
[2]Warm Months/Cold Months

CHENANGO VALLEY STP
ALTERNATIVES 2B, 2C, 3B and 3C

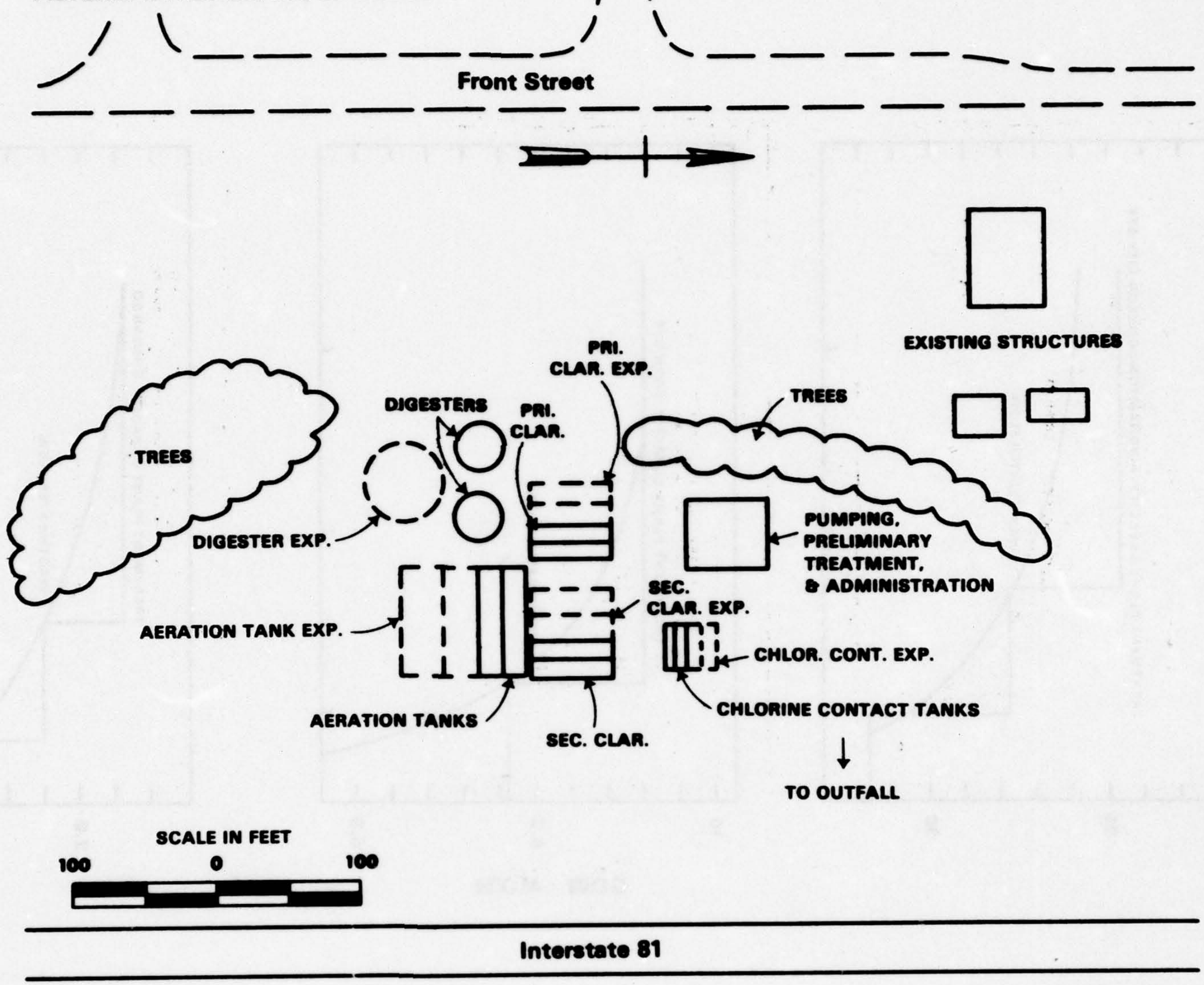
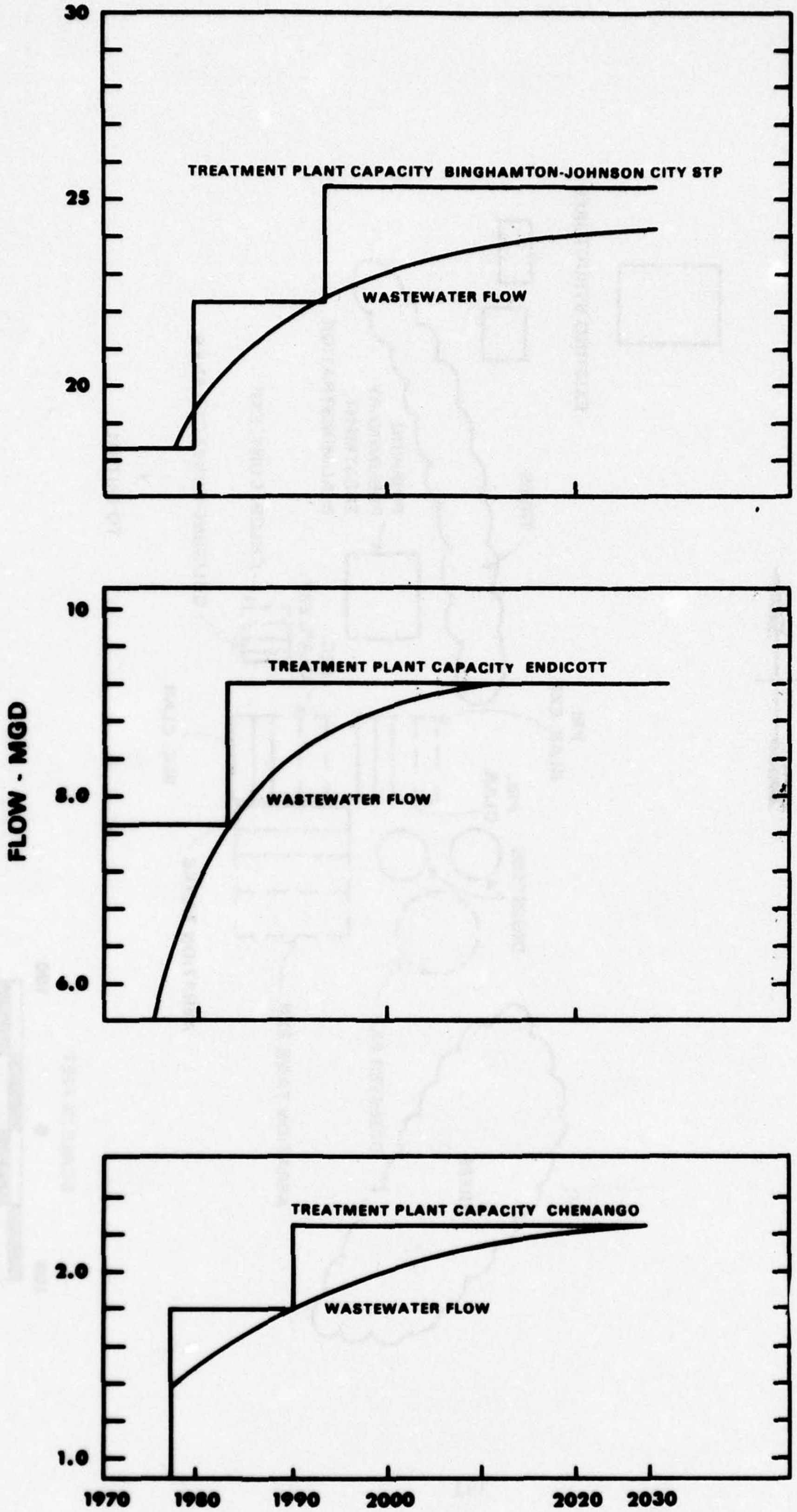
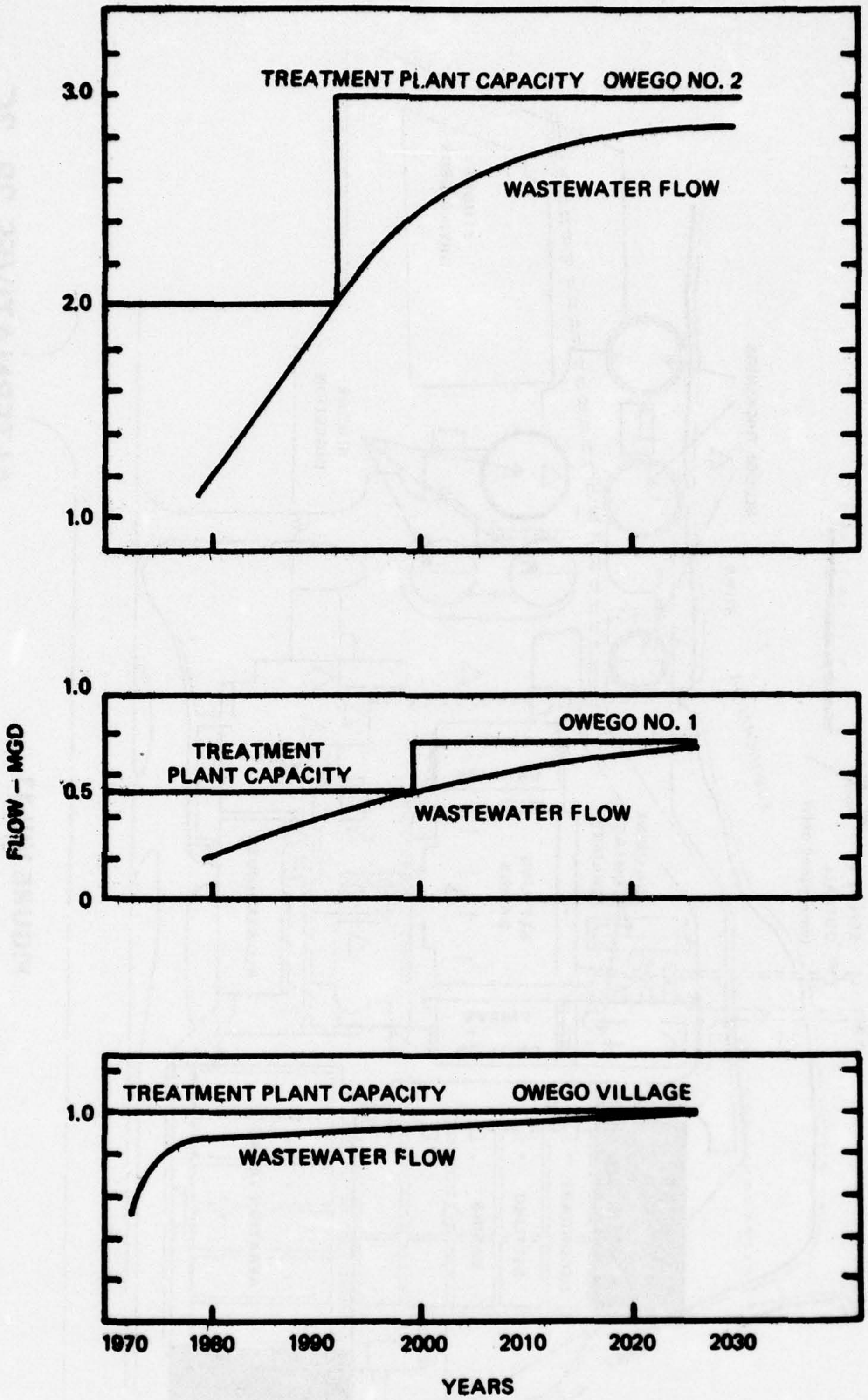


FIGURE VIII-10



WASTEWATER FLOWS AND TREATMENT PLANT CAPACITIES - BROOME COUNTY

PLAN 2B **FIGURE VIII-11**



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – TIOGA COUNTY

FIGURE VIII-12

PLANS 2A, 2B, 2C, 3A, 3B, and 3C

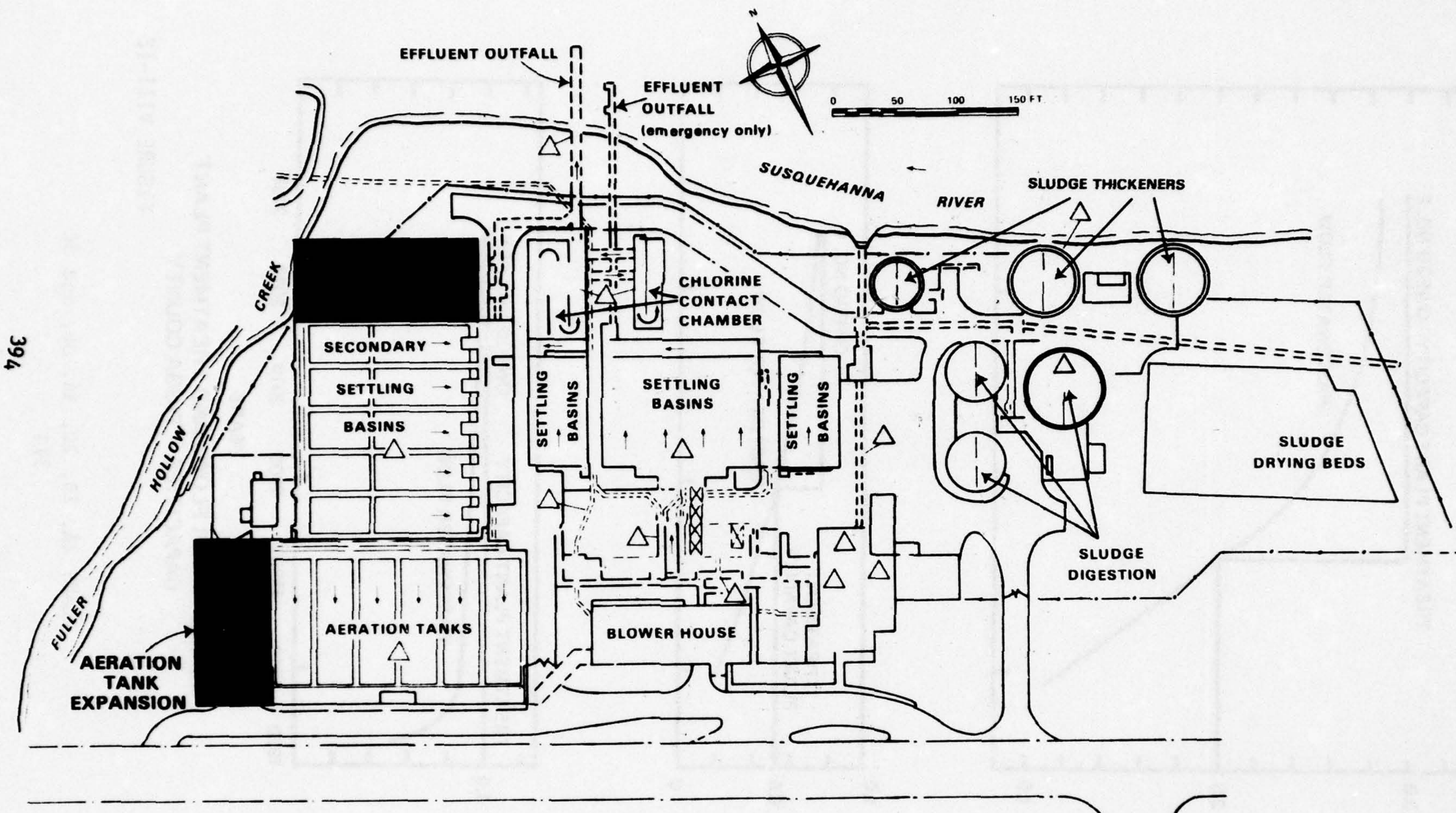
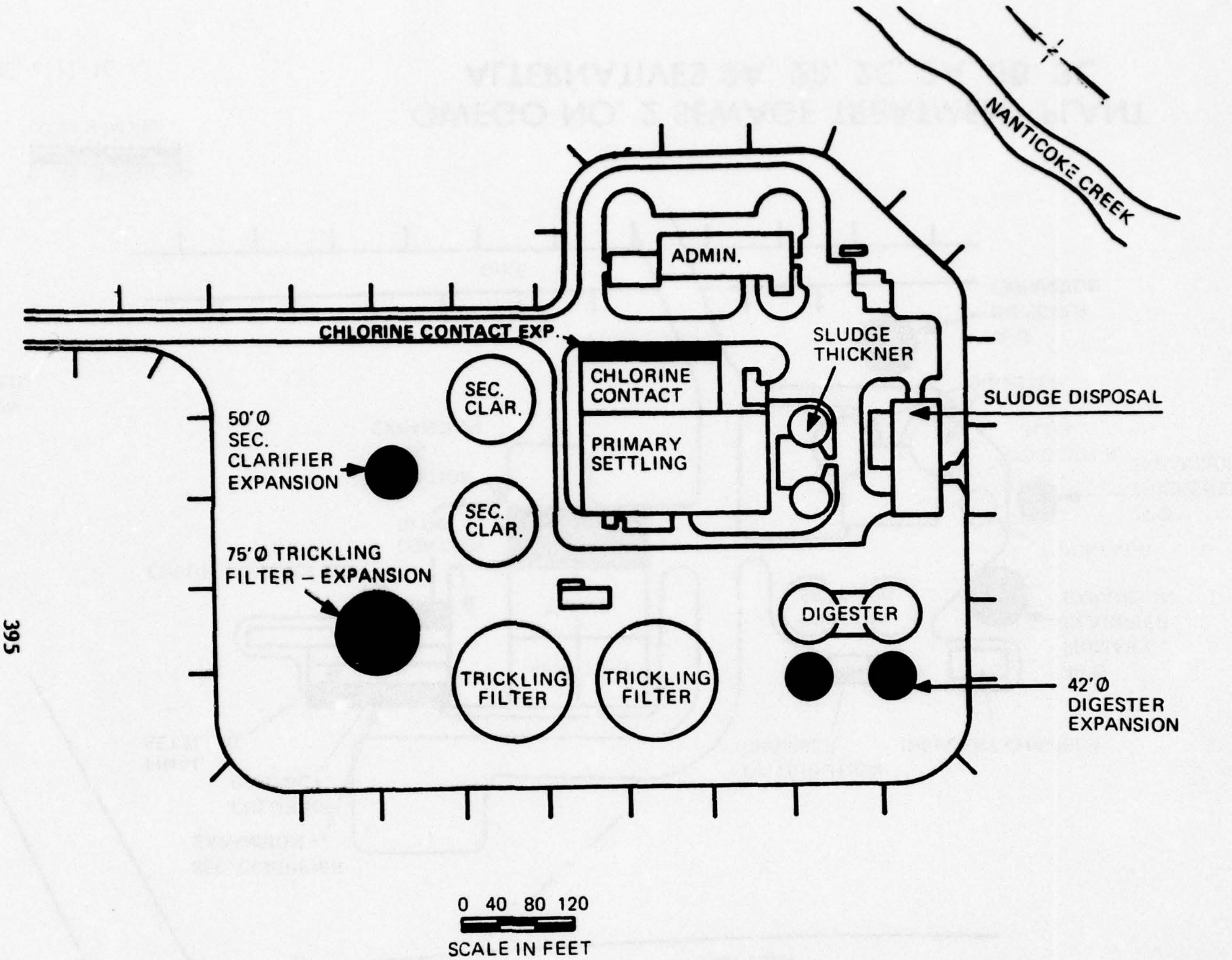


FIGURE VIII-13

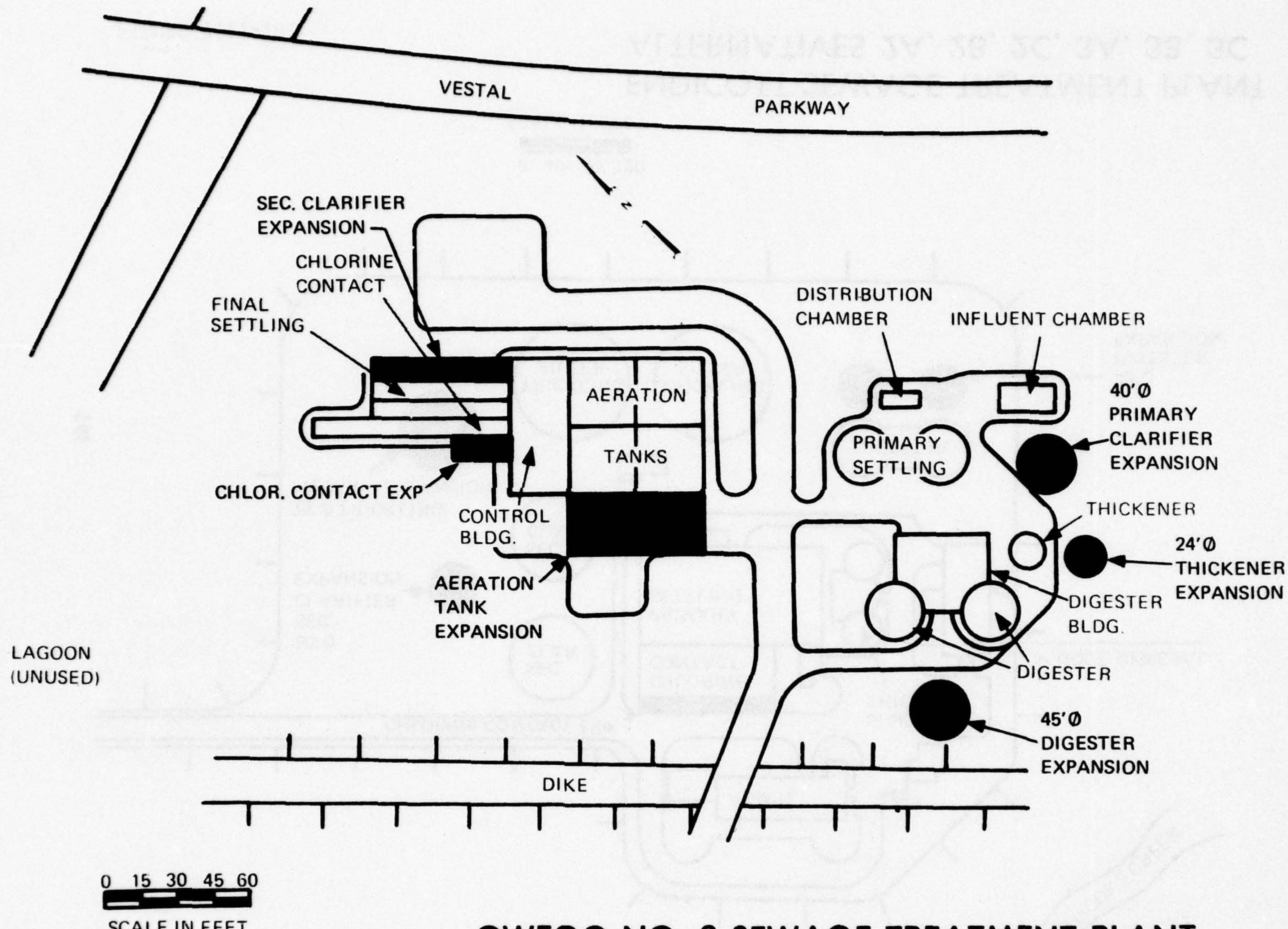
BINGHAMTON-JOHNSON CITY STP

ALTERNATIVES 2B, 2C



ENDICOTT SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-14



OWEGO NO. 2 SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-15

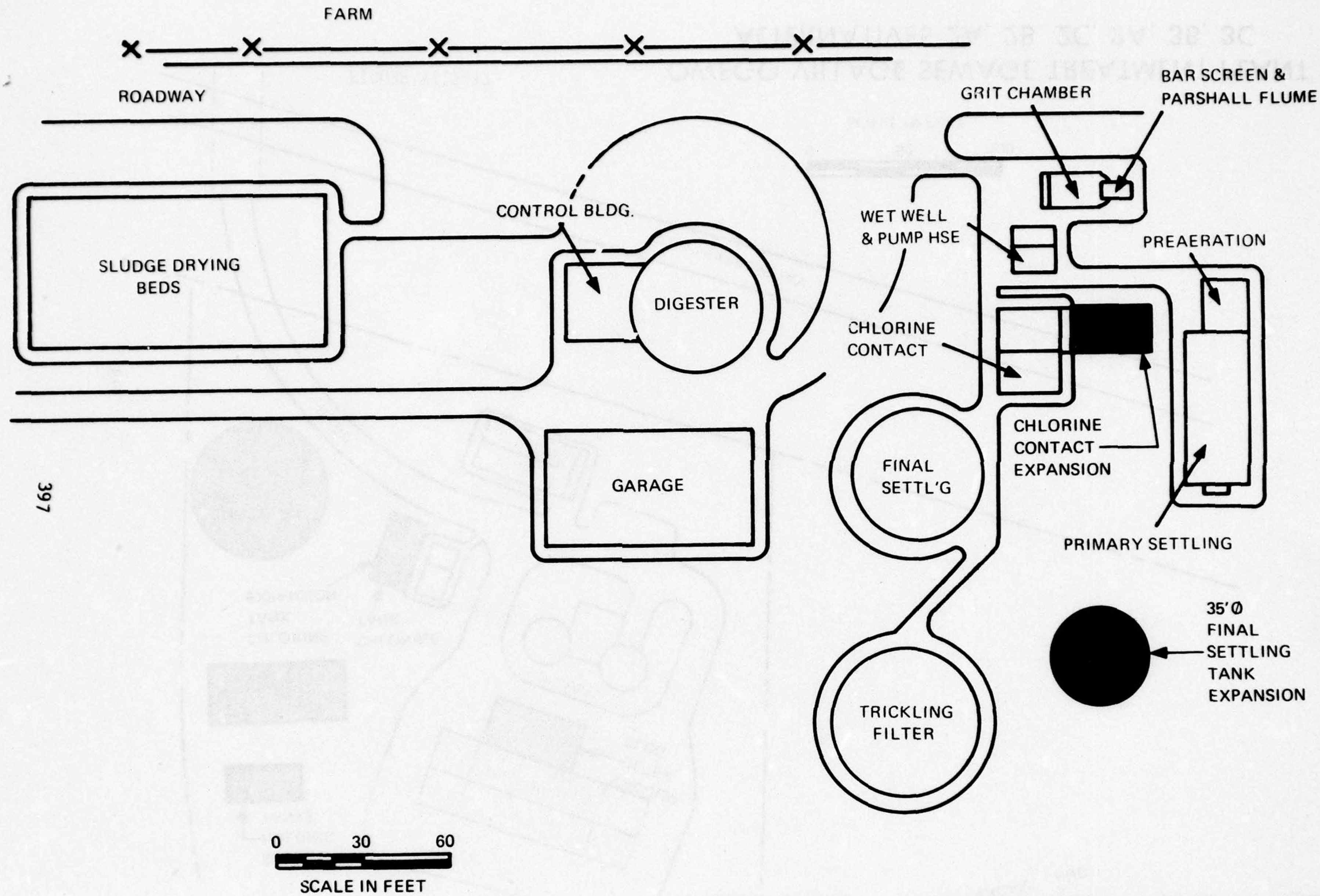


FIGURE VIII-16

OWEGO NO. 1 SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

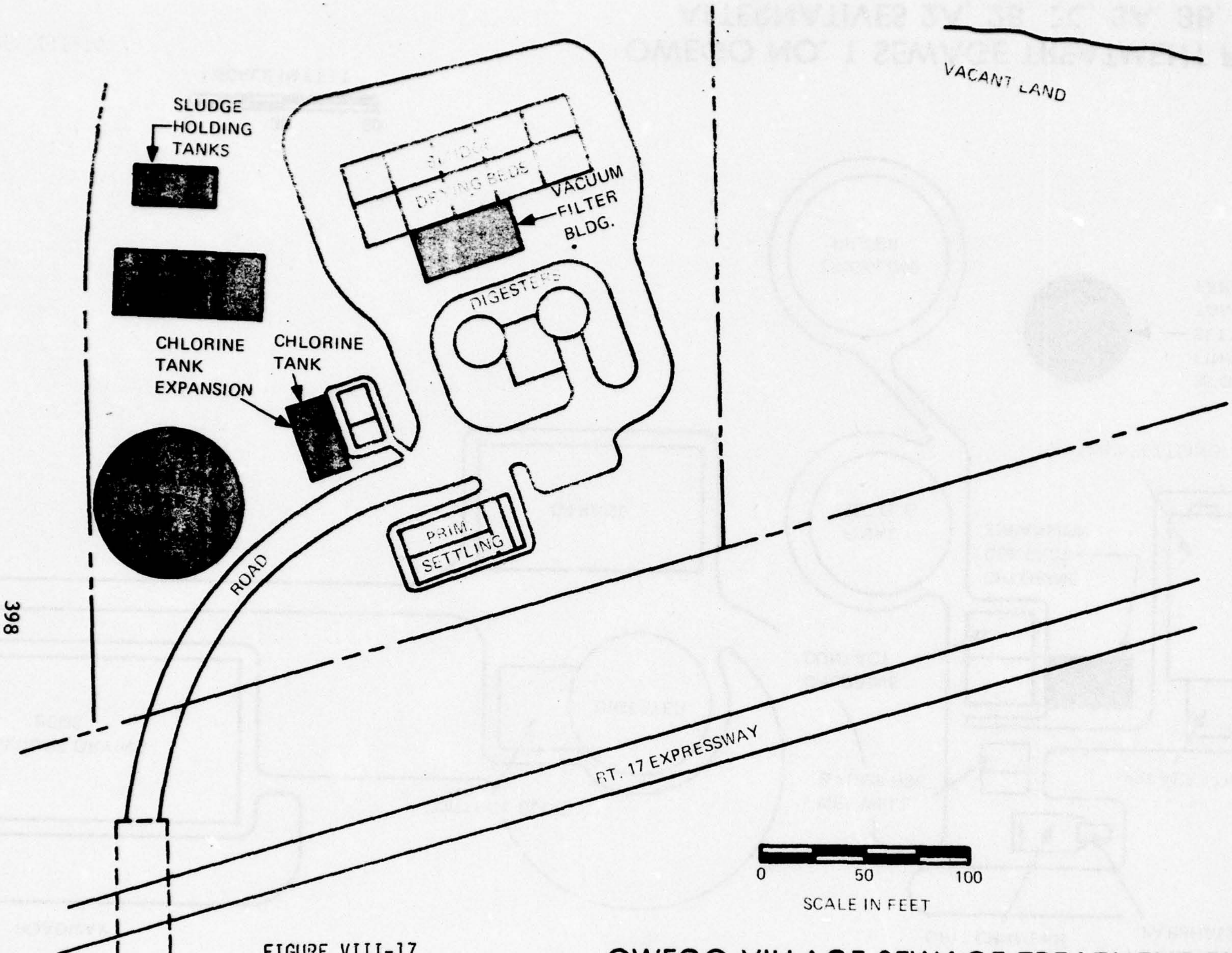


FIGURE VIII-17

OWEGO VILLAGE SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

TABLE VIII-18

PLAN 2B
CONSTRUCTION SCHEDULE

	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth [1] ($ Million)	Description
Binghamton-Johnson City	1.0	1977	0.21		Infiltration control
	39.5	1977	3.58		Storm overflow
	29.0	1977	0.05		Raw wastewater pumping
	22.2	1977	0.71		Aerator and clarifier
	25.4	1991	0.71		Aerator and clarifier
				4.9	
Endicott	0.8	1977	1.57		Nanticoke Valley Interceptor
	9.2	1983	1.95		Additional Secondary Treatment capacity
				2.9	
East Owego	3.0	1992	1.26		General Expansion
				0.5	
West Owego	0.7	2000	0.47		General Expansion
				0.1	
Owego Village	1.0	1977	1.02		New Secondary Treatment capability
	3.0	1977	0.42		Micro-screening
		1977	0.58		Inflow control
				2.0	
Chenango Valley	1.7	1977	1.90		New Secondary Treatment plant
	2.2	1985	.46		General Expansion
				2.2	
Total Present Worth				12.6	

[1] 50 years @ 6-1/8%

TABLE VIII-19

CAPITAL COST BREAKDOWN FOR AN AERATION TANK AND SECONDARY CLARIFIER SET AT THE BINGHAMTON–JOHNSON CITY STP*

1)	Clarifier	$190,000
2)	Aeration Tank (30′ X 100′)	225,000
3)	Yard Piping & Renovation Instrumentation	80,000
4)	Additional Pumping and Valves	50,000
	Sub Total	545,000
	Eng. & Cont. @ 30%	164,000
	Total	$709,000

*July 1975 ENR 2248

TABLE VIII-20

CAPITAL COST BREAKDOWN FOR EXPANSION OF THE SECONDARY TREATMENT FACILITIES AT ENDICOTT STP FROM 7.7 TO 9.2 MGD CAPACITY*

1)	Preliminary Treatment	44,000
2)	Raw Wastes Pumps	74,000
3)	Trickling Filter (75'ϕ - 6'deep)	105,000
4)	Secondary Clarifier (50'ϕ X B'- 6 SWD)	158,000
5)	Chlorine Tank & Feed	51,000
6)	Digester Incl Bld Co. (42'ϕ - 30"high)	440,000
7)	Sludge Pumping	84,000
8)	Piles (for all structures)	345,000
9)	Misc. Electricity, Heating and Ventilation	148,000
10)	Yard Piping	54,000
	Subtotal	1,503,000
	Engineering & Contingencies @ 30%	451,000
	Total	1,954,000

*July 1975 ENR 2248.

TABLE VIII-21

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 2 CAPACITY FROM 2.0 MGD
TO 3 MGD*

1)	Prelimination Treatment & R.W. Pumps, wet well	137,000
2)	Preliminary Settling Tanks 1 tank – 1250 SF	63,000
3)	Aeration Tank & Air System 31,000 CF	116,000
4)	Pumps – Recirculating Piping, Structure Upgrade	80,000
5)	Sludge Thickener (for 4 MGD 24′ Diameter)	42,000
6)	Digester 45′ ϕ X 20′ Diameter	220,000
7)	Chlorine Tank & Feed System	58,000
8)	Sludge Pumping	47,000
9)	Final Clarifiers 55′ X 12′ X 8.5′ depth	74,000
10)	Misc. – Electricity, Heating and Ventilation Sitework	132,000
	Subtotal	969,000
	30% Engr & Cont	291,000
	Total	1,260,000

*July 1975 ENR 2248.

TABLE VIII-22

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 1 CAPACITY FROM 0.5 MGD
TO 0.7 MGD*

1)	Preliminary Treatment	26,000
2)	R.W. Pumps, Internal Piping	90,000
3)	Final Settling Tank (35′ϕ, Dept = 8′)	74,000
4)	Chlorination Tank (16′ X 40′ X 6′)	47,000
5)	Misc. Electrical Instrumentation Heating and Vent.	58,000
6)	Yard Piping, Effluent Piping	68,000
	Subtotal	363,000
	30% Engr. & Cont.	109,000
	Total	472,000

*July 1975 ENR 2248.

TABLE VIII-23

CAPITAL COST BREAKDOWN FOR UPGRADING THE VILLAGE OF OWEGO STP SECONDARY TREATMENT CAPACITY TO 1 MGD*

1) Secondary Pump Station (Max Flow = 3.0)	200,000
2) Trickling Filter A = 4,370 SF 75′ϕ X 5′-6″ Stone Depth	87,000
3) Final Settling TKS A = 1,904 SF 2 - 14′ X 68′ X 7′SWD	78,000
4) Chlorine Tanks Vol = 3,800 16′ X 40′ X 6′ SWD	25,000
5) Sludge Holding Tanks 2 - 19′ X 22′ X 9′SWD With Mixers	60,000
6) Vacuum Filter Area = 100 SF including Building 8′ϕ X 4′ Face	160,000
7) Preliminary Treatment – Screens, Grit removal Conninutor, etc.	65,000
8) Site Work – Added Fill, Yard Piping, Grading, etc.	110,000
Subtotal	785,000
30% Engr. & Cont.	235,000
Total	1,020,000

*July 1975 ENR 2248.

TABLE VIII-24

CAPITAL COST BREAKDOWN FOR SECONDARY TREATMENT PLANT AT CHENANGO VALLEY (Capacity of 1.7 mgd)*

1.	Preliminary Treatment	$ 119,000
2.	Primary Settling Tanks (A = 2145 SF 3 - 13' x 55' x 8' SWD)	108,000
3.	Aeration Tanks (V = 57,600 CF 3 - 20' x 80' x 12' SWD)	146,000
4.	Diffused Aeration System	246,000
5.	Secondary Settling Tanks (A = 2436 SF 3 - 14' x 58' x 10' SWD)	116,000
6.	Sludge Digestion (V = 68,000 CF 2 - 46'ϕ)	360,000
7.	Chlorine Contact Tanks (V = 5880 CF 3 - 8' x 35' x 7' SWD)	37,000
8.	Chlorine Feed Equipment	40,000
9.	Yardwork	175,000
10.	Outfall to Chenango River	115,000
		$1,462,000
	30% Engr. & Cont.	438,000
	TOTAL	$1,900,000

*July 1975 ENR 2248

TABLE VIII-24 (Continued)

(Expansion of STP Capacity from 1.7 mgd to 2.2 mgd)

1.	Preliminary Treatment	$ 29,000
2.	Primary Settling Tank (A = 605 SF 1 = 11' x 55' x 8' SWD)	33,000
3.	Aeration Tank (V = 16,300 CF 1 - 17' x 80' x 12' SWD)	30,000
4.	Diffused Aeration Tank	55,000
5.	Secondary Settling Tank (A = 696 SF 1 - 12' x 58' x 10' SWD)	35,000
6.	Sludge Digestion (V = 16,000 CF 1 - 32' ϕ)	129,000
7.	Chlorine Contact Tank (V = 1715 CF 1 - 7' x 35' x 7' SWD)	9,000
8.	Chlorine Feed Equipment	13,000
9.	Yardwork	21,000
		$354,000
	+ 30% Engr. & Cont.	106,000
	TOTAL	$460,000

TABLE VIII-25

CAPITAL COST BREAKDOWN FOR THE NANTICOKE INTERCEPTOR (AVERAGE DESIGN FLOW = 0.8 M.G.D.* PEAK DESIGN FLOW = 2.3 M.G.D.)

1)	18" sewer – 25,000' @ $38/LF	957,000
2)	110 manholes @ $1,100	120,000
3)	Misc. items	33,000
4)	Railroad crossing 27" casing 100' @ $270	27,000
5)	Creek Crossing 200/LF @ $350/LF	70,000
	Subtotal	1,207,000
	30% Engr. & Contr.	362,000
	Total	1,569,000

*use 18" ϕ sewer @ S = 0.0012 min. assume avg. depth = 12-14 feet.

July 1975 ENR 2248.

TABLE VIII-26

PLAN 2B
OPERATION AND MAINTENANCE COSTS

	First Year	Last Year	Description	Annual Cost ($ Million/Yr)	Present Worth[1] ($ Million)
Binghamton-	1977	2026	Secondary	0.72	
Johnson City	1977	2026	Storm Overflow	0.05	
					11.9
Endicott	1977	1982	Secondary	0.31	
	1983	2026	Secondary	0.36	
					5.1
Chenango	1977	1984	Secondary	0.13	
	1985	2026	Secondary	0.15	
					2.1
East Owego	1977	1991	Secondary	0.14	
	1992	2026	Secondary	0.19	
					2.4
West Owego	1977	1989	Secondary	0.04	
	2000	2026	Secondary	0.05	
					0.6
Owego Village	1977	2026	Secondary	0.07	
	1977	2026	Microscreening	0.002	
					1.1
				Total Present Worth:	23.2

[1] 50 years @ 6-1/8%.

TABLE VIII-27

PLAN 2B

CAPITAL COSTS (MILLION $) AND OPERATION AND MAINTENANCE COSTS ($$10^3$/Yr)

Year	Binghamton Johnson Capital	Binghamton Johnson O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	E. Owego Capital	E. Owego O&M	W. Owego Capital	W. Owego O&M	Owego Village Capital	Owego Village O&M
1977	4.55	770	1.90	130	1.57	310		140		40	2.0	70
1983					1.95	360						
1985			.46	150								
1989												
1990												
1991	.71											
1992							1.26	190				
1995												
2000									.47	50		
2010												
2018												
2026		770		150		360		190		50		70
Present Worth Million $	4.9	11.9	2.2	2.1	2.9	5.1	.5	2.4	.1	.6	2.0	1.1
Present Worth of Replacement Million $	6.1		0.5		3.8		0.8		0.2		0.5	
Total Present Worth Million $	22.9		4.8		11.8		3.7		0.9		3.6	

TABLE VIII-28

PLAN 2B COSTS – PRESENT WORTH[1]
(MILLION $'s)

	Chenango Valley	Binghamton-Johnson City	Endicott	East Owego	West Owego	Owego Village	Total
Wastewater Treatment							
Capital	1.7	1.1	1.3	0.5	0.1	1.0	5.7
O & M	1.5	7.3	4.4	1.9	0.4	0.7	16.2
Replacement	0.4	4.9	3.6	0.7	0.1	0.3	10.0
Total	3.6	13.3	9.3	3.1	0.6	2.0	31.9
Interceptors							
Capital	–	–	1.6	–	–	–	1.6
O & M	–	–	–	–	–	–	–
Replacement	–	–	–	–	–	–	–
Total	–	–	1.6	–	–	–	1.6
Storm Overflows							
Capital	–	3.6	–	–	–	1.0	4.6
O & M	–	0.8	–	–	–	0.1	0.9
Replacement	–	0.9	–	–	–	0.1	1.0
Total	–	5.3	–	–	–	1.2	6.5
Infiltration Control							
Capital	–	0.2	–	–	–	–	0.2
O & M	–	–	–	–	–	–	–
Replacement	–	–	–	–	–	–	–
Total	–	0.2	–	–	–	–	0.2
Sludge							
Capital	0.465	0.05	0.01	0.005	0.001	0.002	0.5
O & M	0.6	3.8	0.7	0.5	0.2	0.3	6.1
Replacement	0.1	0.3	0.2	0.1	0.1	0.1	0.9
Total	1.2	4.1	0.9	0.6	0.3	0.4	7.5
Totals							
Capital	2.2	4.9	2.9	0.5	0.1	2.0	12.6
O & M	2.1	11.9	5.1	2.4	0.6	1.1	23.2
Replacement	0.5	6.1	3.8	0.8	0.2	0.5	11.9
Total Plan Cost	4.8	22.9	11.8	3.7	0.9	3.6	47.7

[1] Based on a 50 year economic life @ 6-1/8% and ENR = 2248.

PLAN 2C

The intent of this plan is to maintain minimum daily average and minimum instantaneous DO levels above 4 mg/l in the Susquehanna River during both low flow and storm overflow conditions, respectively. Plans 2A, 2B, and 2C are similar except for the degree of regionalization.

MUNICIPAL WASTEWATER MANAGEMENT

Regionalization of STP's

The Broome County plants would be the existing Binghamton-Johnson City and Endicott STP's and a new STP for a limited Chenango Valley service area. The first phase of this plan would initially serve a limited Chenango Valley wastewater management area. In the final phase, the treatment plant and service area would be expanded to include the entire wastewater management area. Plan 2C is identical to Plan 2B except the scope of sewerage service for Chenango Valley would be limited to a smaller area for 5 years. Initial treatment plant capacity in 1977 would be 1.0 mgd (vs. 1.7 mgd for Plan 2B). After 1982, the entire Chenango Valley service area would be sewered and the STP expanded to 2.2 mgd.

Tioga County plants would be the Town of Owego STP's No. 1 (West Owego) and 2 (East Owego), and the Village of Owego STP (all existing).

Service areas and interceptors included in this plan are shown in Plate 5. With the exception of the Chenango Valley sewerage system, these are the same interceptors included in the Baseline Plan. Plate 6 details the phasing of the Chenango Valley service area.

Treatment Levels and Processes

Secondary treatment would be applied at all STP's. The existing activated sludge processes would be used at the Binghamton-Johnson City STP and at the Town of Owego STP No. 2. A new activated sludge treatment plant would be built at Chenango. The existing trickling filter processes would be used at the Endicott STP and the Town of Owego STP No. 1. The Village of Owego STP was assumed

to be upgraded to secondary treatment using a trickling filter process (recently changed to a proposal for an activated sludge process).

Infiltration Control Level in the City of Binghamton

For the level of treatment (secondary) and the regionalization scheme assumed in this plan, an infiltration reduction of 1 mgd was found to be economically justifiable.

Nonstructural Measures for Flow Reduction

In this plan, the Village of Endicott and the Town of Owego would institute metered user rates. The increase in price the consumer would be paying should result in a reduction of water consumption, and, therefore, wastewater flows.

An educational program to encourage the use of water saving devices could result in an estimated 20 percent reduction in the projected incremental increase in per capita flows. Although such a program could be undertaken, no formal educational activities have been specified in this plan.

STORM OVERFLOW MANAGEMENT

Five microscreening units followed by chlorination would be provided near five overflow sites in the City of Binghamton (see Figure VIII-18). This system would maintain a minimum instantaneous DO of 4 mg/l during most storm conditions. A detailed description of the overflow treatment system is given in Chapter VII. An estimate was also made of the costs of inflow control and microscreening treatment for the Village of Owego's combined sewer system.

SLUDGE MANAGEMENT

Several sludge management techniques have been analyzed in Chapter VI. The land application of liquid sludge is recommended, with landfill of dewatered sludge provided as a backup.

PERFORMANCE

Impact on Susquehanna River dissolved oxygen levels:

1. The minimum daily average dissolved oxygen level would be between 4 and 5 mg/l during low river flow and dry wastewater flow conditions.

2. The minimum instantaneous dissolved oxygen level would be between 4 and 5 mg/l during design storm conditions.

The pollutant mass loads discharged to the Susquehanna and Chenango Rivers by the STP's proposed in this alternative are summarized in Table VIII-29.

CONSTRUCTION SCHEDULE

The Chenango Valley phased treatment plant would be built by 1977 to conform with the Federal requirements for secondary treatment by utilizing an activated sludge process (see Figure VIII-19). The plant would have an initial capacity of 1.0 mgd to be expanded to 2.2 mgd by 1982, with sewering extending the entire Chenango Valley service area.

Expansion of the wastewater facilities would be required during the planning period at all STP's to handle the increasing flow expected (see Figures VIII-20 and VIII-21).

At the Binghamton-Johnson City STP, the raw wastewater pumping capacity would be increased from 18.3 mgd to 29 mgd by 1977. Also by this year, one aerator and clarifier unit will be added to increase the capacity from 18.3 mgd to 22.2 mgd, with another aeration and clarifier unit to be added by the year 1991 to increase the capacity from 22.2 mgd to 25.4 mgd (see Figure VIII-22).

Storm overflow control facilities are assumed to be constructed at the City of Binghamton by 1977. Five microscreening facilities having a total treatment capacity of 39.5 mgd will be located as shown in Figure VIII-18.

At Endicott STP, the secondary treatment units will need expansion from 7.7 mgd to 9.2 mgd capacity by 1983. The trickling filters, secondary clarifitions, digesters, chlorination tank, raw wastewater and sludge pumping capacities will be expanded (see Figure VIII-23).

The Nanticoke service area wastewater will be transmitted to the Endicott STP through an 18 inch diameter sewer that has a design average flow of 0.8 mgd and a peak flow of 2.3 mgd. The sewer length is 25,000 feet. This interceptor will be built by 1977.

At the Town of Owego STP No. 2, the plant treatment capacity will be upgraded from the present 2.0 mgd to 3.0 mgd by 1992. The additional units include preliminary treatment; a primary settling tank with a 1,250 square foot capacity; a 31,000 cubic foot aeration tank and air system; a sludge thickener for 4 mgd; a digester 45' in diameter by 20' in depth; a chlorination tank 25' by 18' and 8' deep; and two final clarifiers having a 3 mgd capacity, each being 55' by 12' and 8.5' deep (see Figure VIII-24).

At the Town of Owego STP No. 1, the plant capacity will be expanded by the year 2000 from the present 0.5 mgd to 0.7 mgd. The added facilities will include a preliminary treatment capacity, a final settling tank, and a chlorination tank (see Figure VIII-25).

At the Village of Owego, the treatment facilities will be upgraded to provide secondary treatment by 1977. The capacity of the upgraded facility will be 1 mgd. The facilities added will include a trickling filter with a surface area of 4,370 square feet, 75' diameter and 5'6" in depth; a secondary pump station designed to handle a maximum flow of 3.0 mgd; two final settling tanks each 14' by 68' and 7' deep; a chlorine tank, volume 3,800 cubic feet; two sludge holding tanks with mixers each 19' by 22' and 9' deep; and a vacuum filter with an area of 100 square feet (see Figure VIII-26).

At the Village of Owego, a microscreening treatment unit with a capacity of 3 mgd will be built by 1977 to provide treatment of storm overflows. Also, steps would be taken to control direct stream inflow into the existing system.

A summary of the construction schedule of wastewater treatment facilities needed at each service area is given in Table VIII-30.

COST ANALYSIS

The capital costs breakdown for the expansion and/or upgrading of the existing treatment facilities at Binghamton-Johnson City, the Village of Endicott, Town of Owego STP No. 1, Town of Owego STP No. 2, and the Village of Owego

are shown in Tables VIII-31 through VIII-35. Tables VIII-36 and VIII-37 break down the capital costs for the new Chenango Valley STP and the Nanticoke interceptor, respectively.

The operation and maintenance costs of the wastewater treatment plants for each service area during the planning period are given in Table VIII-38.

The capital, O&M and replacement costs for each service area are summarized in Table VIII-39. Also shown in that table are the years these expendituures will be incurred. These correspond to the construction schedule described in the previous section. The replacement costs were estimated using an equivalent annual sinking fund approach.

The present worth of all the expenditures during the 50-year planning period is given in Table VIII-40 for each service area. The present worth costs for this plan are: capital -- \$12.6 million, O&M -- \$23.2 million, and replacement -- \$11.9 million, for a total cost of \$47.7 million.

CITY OF BINGHAMTON
LOCATION OF PLANNED OVERFLOW CONTROL FACILITIES

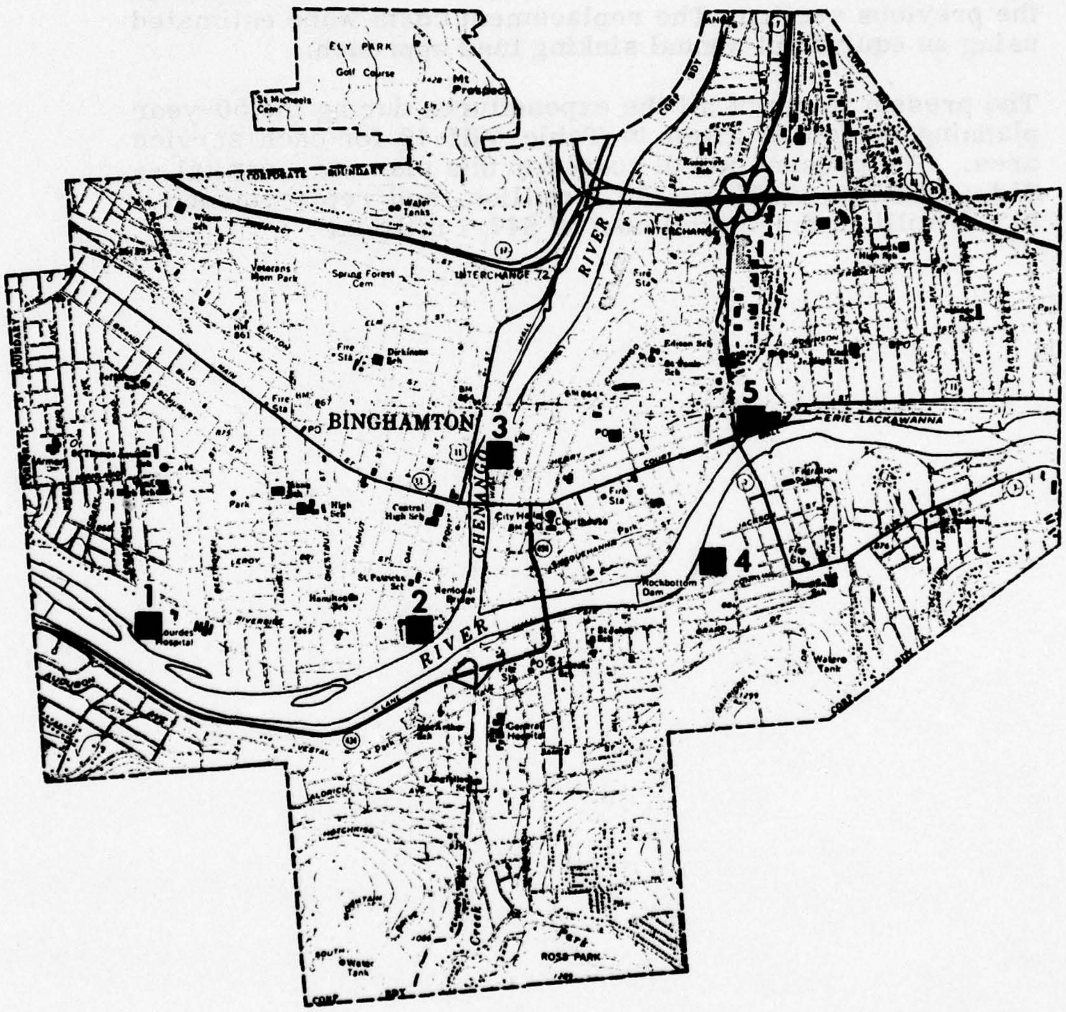


FIGURE VIII-13

TABLE VIII-29

PLAN 2C

AVERAGE DAILY POLLUTION LOADS TO THE SUSQUEHANNA RIVER[1]

Source	Parameter					
	Flow (M.G.D)	SS lb/day	BOD[2] lb/day	NOD[2] lb/day	N lb/day	P lb/day
Binghamton-Johnson	24.2	4200	3100 / 4700	7600 / 12900	3470	870
Endicott	9.2	1470	1470	5200 / 8800	2260	600
Chenango	2.2	400	400	1300 / 2800	710	165
Owego #2	2.8	360	360	1200 / 2500	640	150
Owego #1	.7	250	250	830 / 1400	305	80
Owego Village	.97	125	125	415 / 700	155	40
Total	40.1	6805	5705 / 7305	16545 / 29100	7540	1905

[1]Year 2020
[2]Warm Months/Cold Months

CHENANGO VALLEY STP
ALTERNATIVES 2B, 2C, 3B and 3C

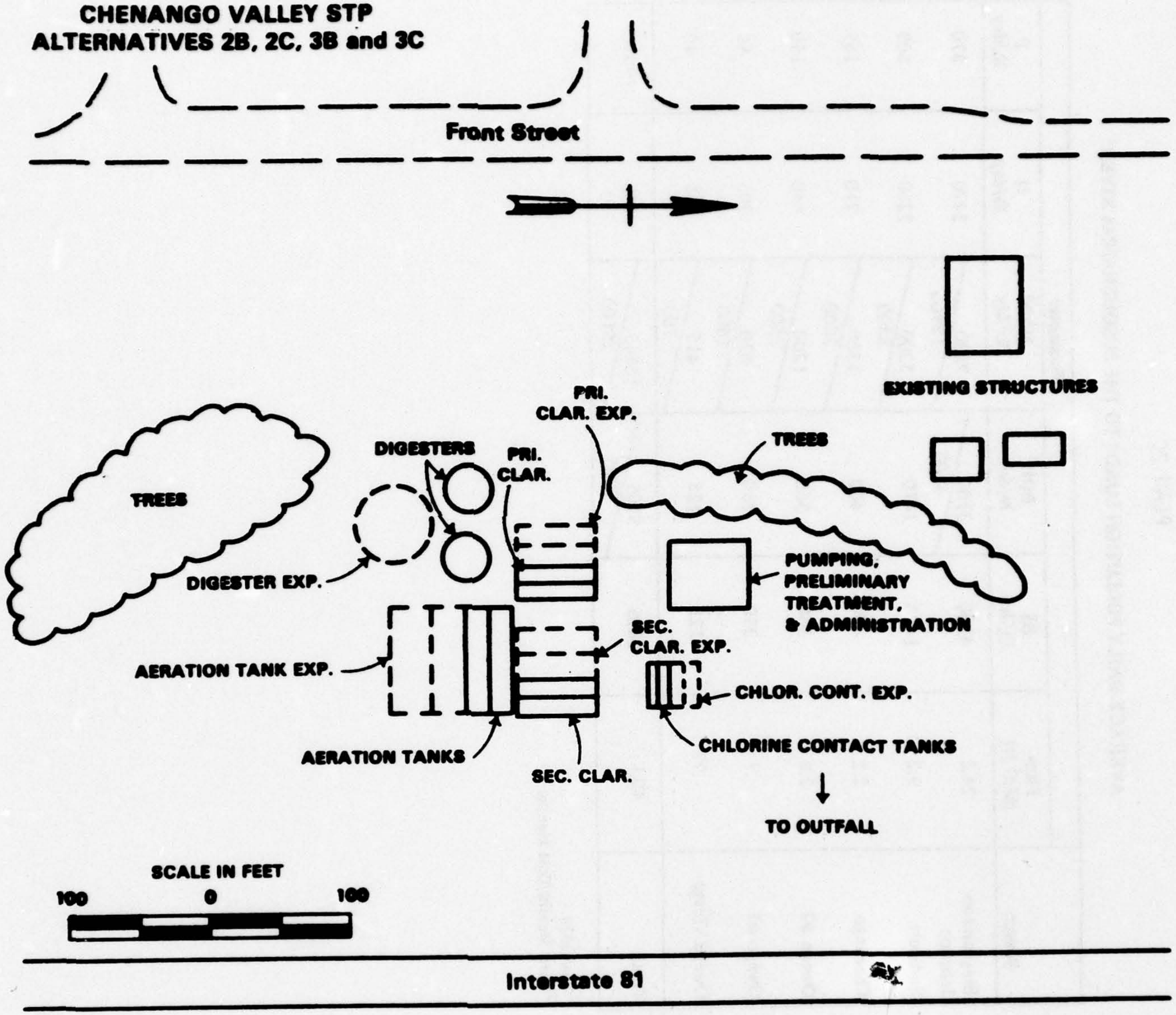
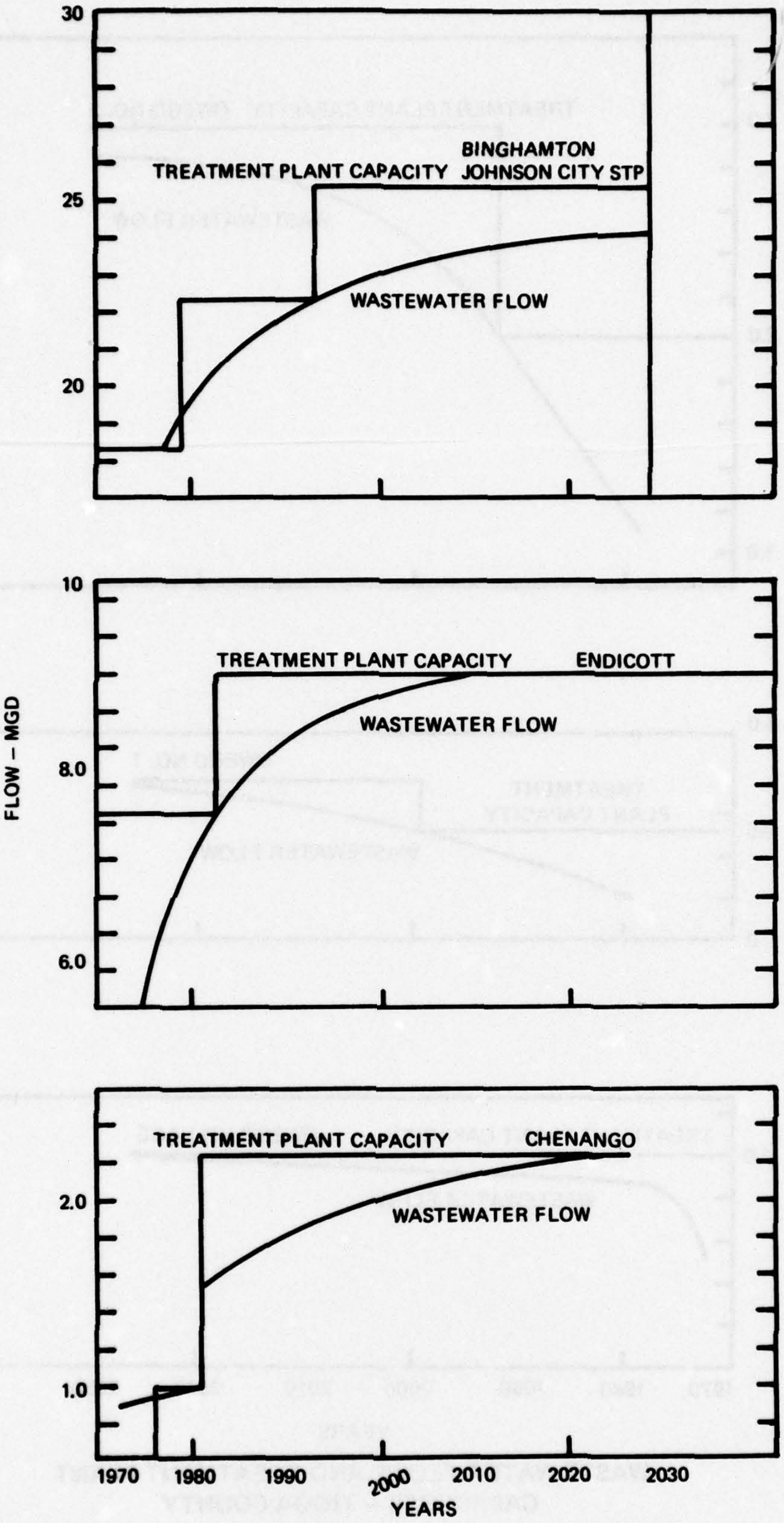


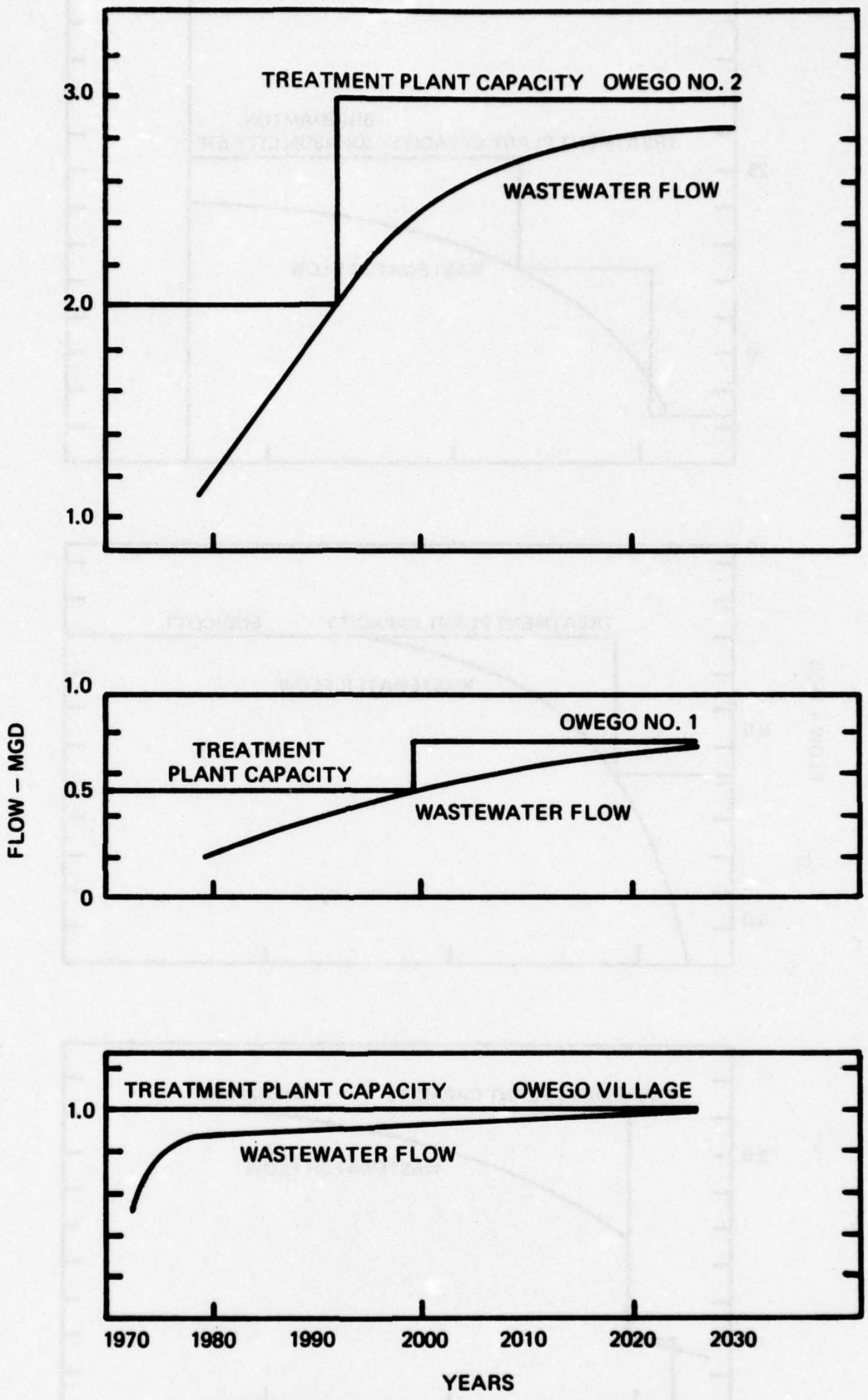
FIGURE VIII-19



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – BROOME COUNTY

FIGURE VIII-20

PLAN 2C



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – TIOGA COUNTY

FIGURE VIII-21

PLANS 2A, 2B, 2C, 3A, 3B, and 3C

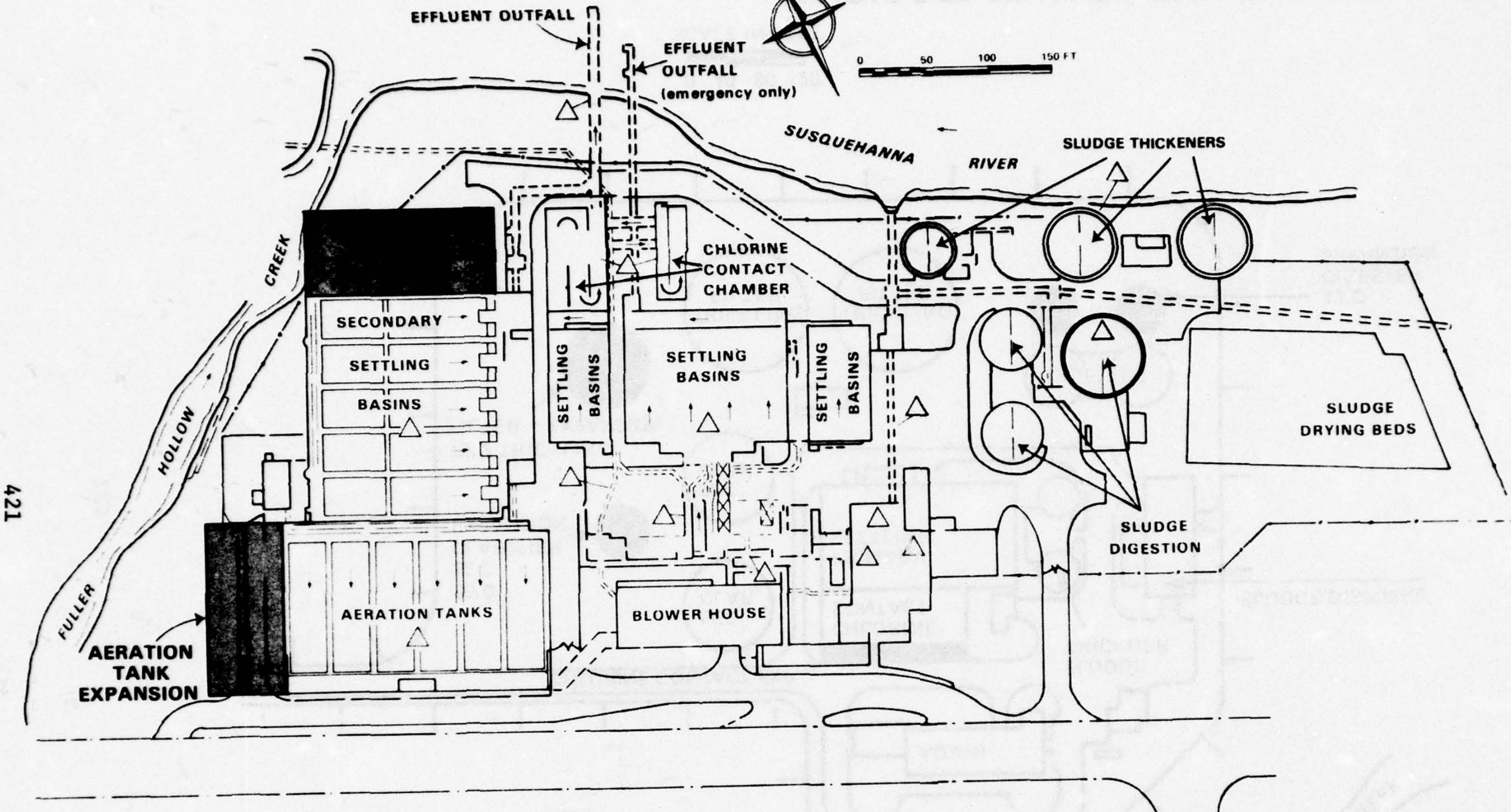
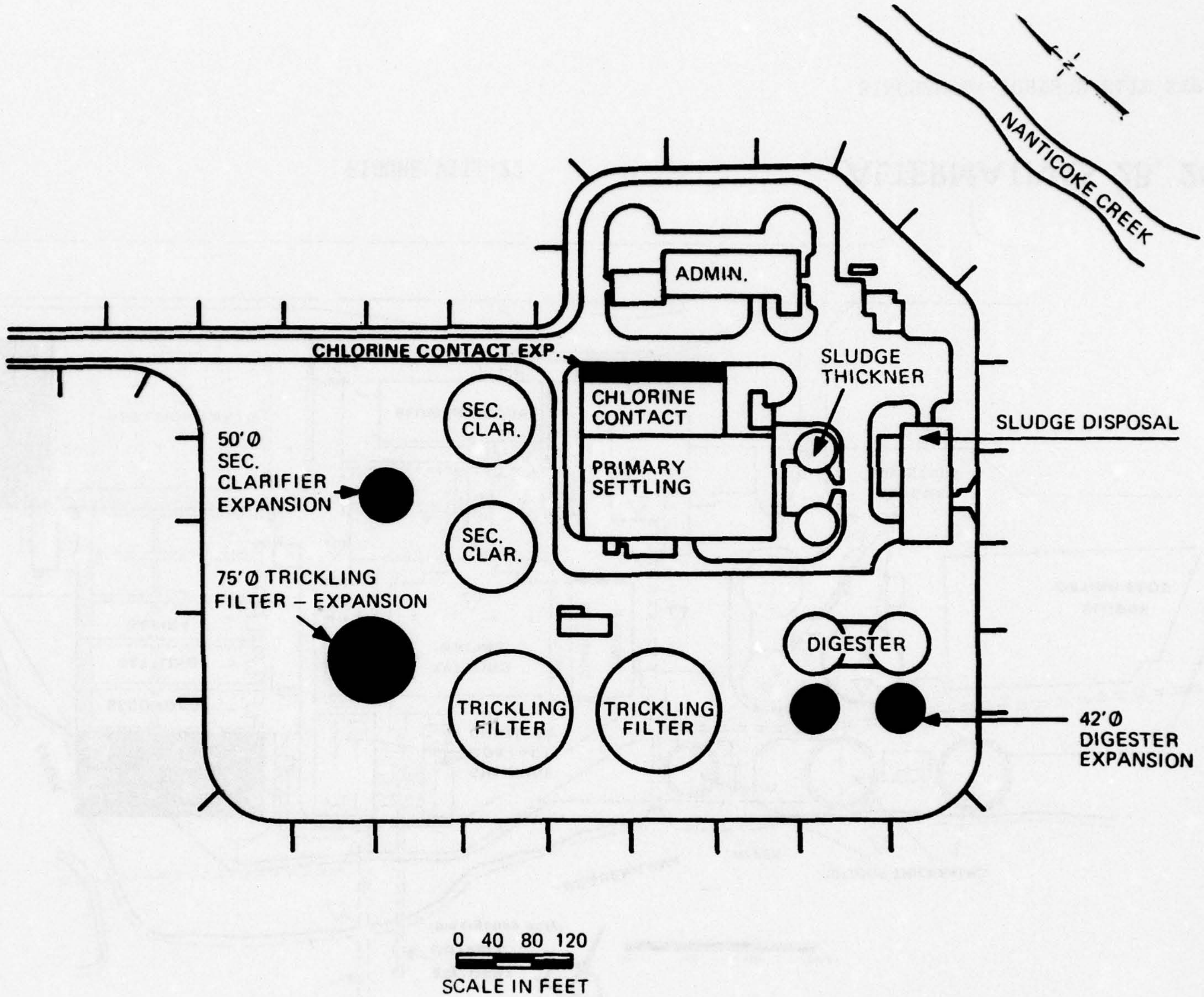


FIGURE VIII-22

ALTERNATIVES 2B, 2C

BINGHAMTON-JOHNSON CITY STP



ENDICOTT SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-23

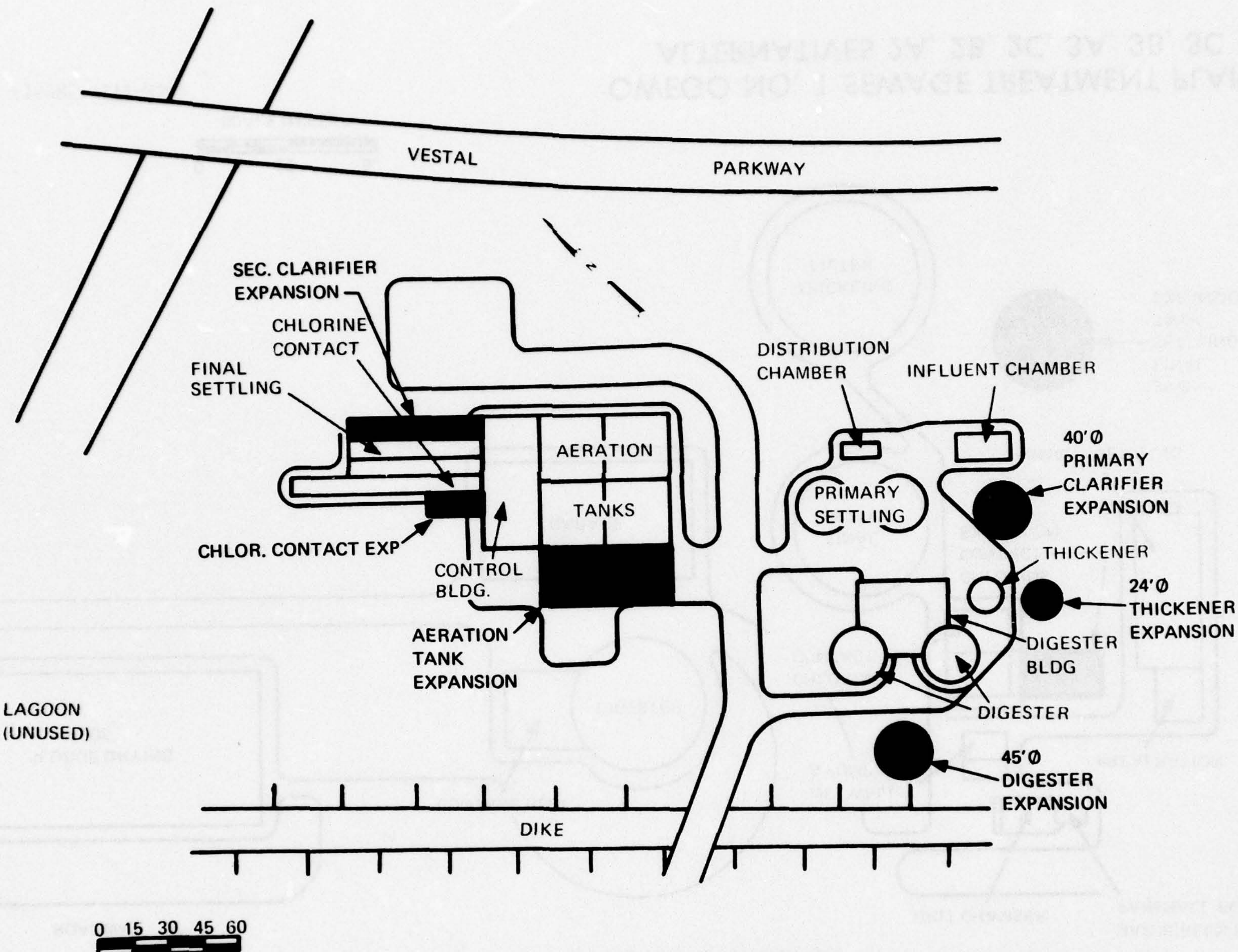


FIGURE VIII-24

OWEGO NO. 2 SEWAGE TREATMENT PLANT ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

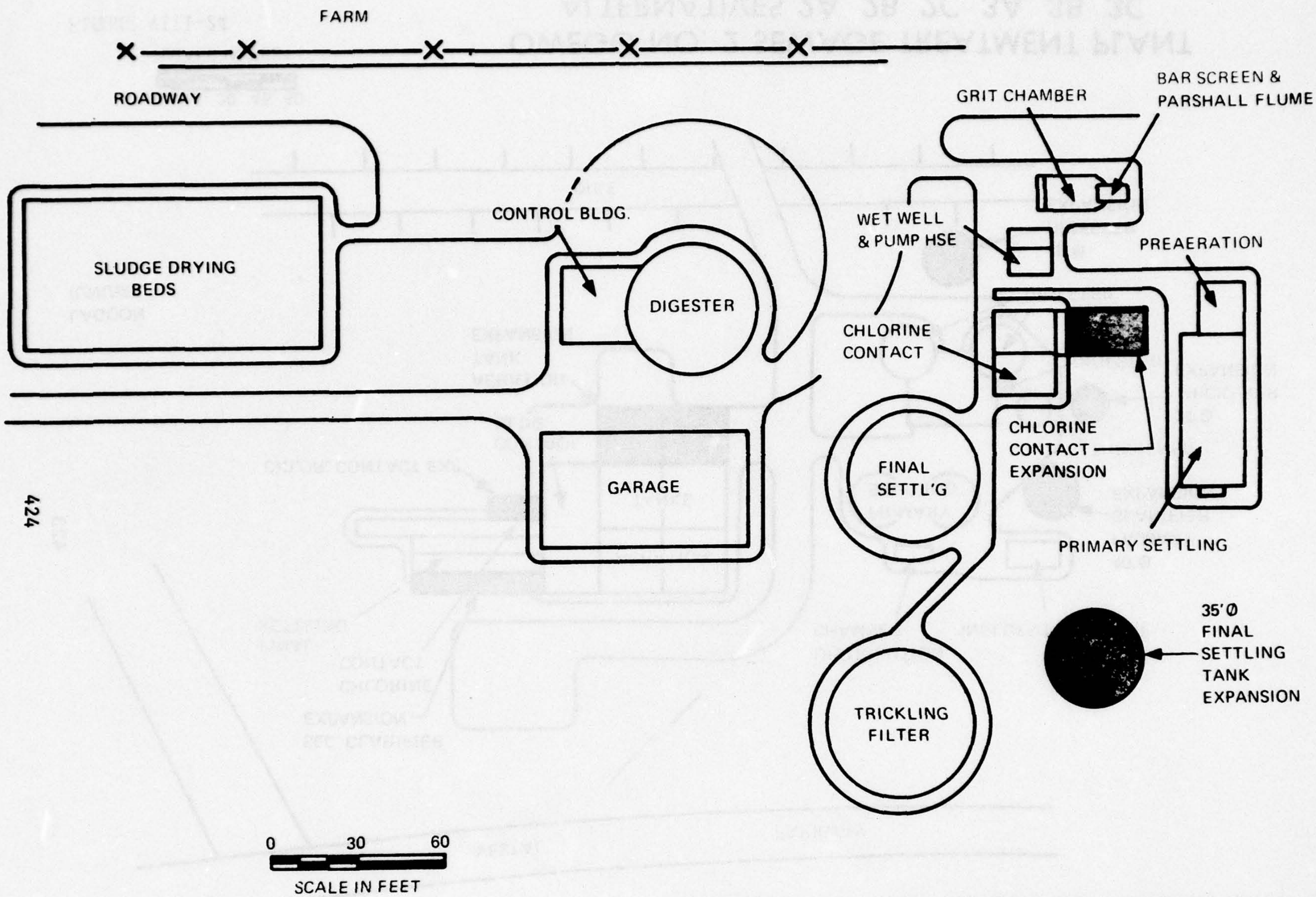
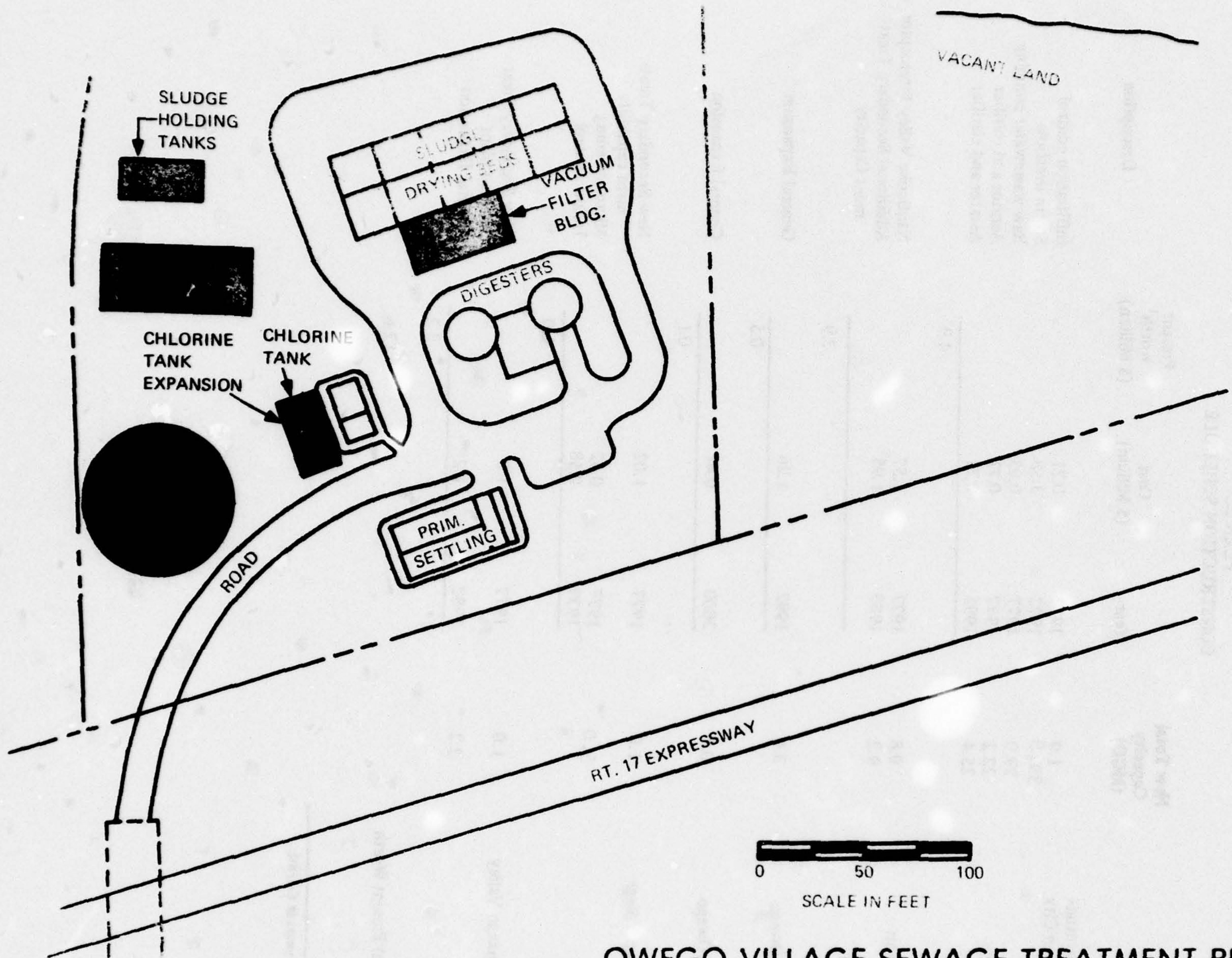


FIGURE VIII-25

OWEGO NO. 1 SEWAGE TREATMENT PLANT ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C



OWEGO VILLAGE SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-26

TABLE VIII-30

PLAN 2C
CONSTRUCTION SCHEDULE

	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[1] ($ Million)	Description
Binghamton-Johnson City	1.0	1977	0.21		Infiltration control
	39,5	1977	3.58		Storm overflow
	29.0	1977	0.05		Raw wastewater pumping
	22.2	1977	0.71		Aerator and clarifier
	25.4	1991	0.71		Aerator and clarifier
				4.9	
Endicott	0.8	1977	1.57		Nanticoke Valley Interceptor
	9.2	1983	1.95		Additional Secondary Treatment Capacity
				2.9	
East Owego	3.0	1992	1.26		General Expansion
				0.5	
West Owego	0.7	2000	0.47		General Expansion
				0.1	
Owego Village	1.0	1977	1.02		New Secondary Treatment capability
	3.0	1977	0.42		Micro-screening
		1977	0.58		Inflow control
				2.0	
Chenango Valley	1.0	1977	1.3		New Secondary Treatment plant
	2.2	1982	1.2		General Expansion
				2.2	
Total Present Worth				12.6	

[1] 50 years @ 6-1/8%

TABLE VIII-31

CAPITAL COST BREAKDOWN FOR AN AERATION TANK AND SECONDARY CLARIFIER SET AT THE BINGHAMTON–JOHNSON CITY STP*

1)	Clarifier	$190,000
2)	Aeration Tank (30′ X 100′)	225,000
3)	Yard Piping & Renovation Instrumentation	80,000
4)	Additional Pumping and Valves	50,000
	Sub Total	545,000
	Eng. & Cont. @ 30%	164,000
	Total	$709,000

*July 1975 ENR 2248

TABLE VIII-32

CAPITAL COST BREAKDOWN FOR EXPANSION OF THE SECONDARY TREATMENT FACILITIES AT ENDICOTT STP FROM 7.7 TO 9.2 MGD CAPACITY*

1)	Preliminary Treatment	44,000
2)	Raw Wastes Pumps	74,000
3)	Trickling Filter (75′ϕ - 6′deep)	105,000
4)	Secondary Clarifier (50′ϕ X B′- 6 SWD)	158,000
5)	Chlorine Tank & Feed	51,000
6)	Digester Incl Bld Co. (42′ϕ - 30″high)	440,000
7)	Sludge Pumping	84,000
8)	Piles (for all structures)	345,000
9)	Misc. Electricity, Heating and Ventilation	148,000
10)	Yard Piping	54,000
	Subtotal	1,503,000
	Engineering & Contingencies @ 30%	451,000
	Total	1,954,000

*July 1975 ENR 2248.

TABLE VIII-33

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 2 CAPACITY FROM 2.0 MGD
TO 3 MGD*

1)	Prelimination Treatment & R.W. Pumps, wet well	137,000
2)	Preliminary Settling Tanks 1 tank – 1250 SF	63,000
3)	Aeration Tank & Air System 31,000 CF	116,000
4)	Pumps – Recirculating Piping, Structure Upgrade	80,000
5)	Sludge Thickener (for 4 MGD 24' Diameter)	42,000
6)	Digester 45' ϕ X 20' Diameter	220,000
7)	Chlorine Tank & Feed System	58,000
8)	Sludge Pumping	47,000
9)	Final Clarifiers 55' X 12' X 8.5' depth	74,000
10)	Misc. – Electricity, Heating and Ventilation Sitework	132,000
	Subtotal	969,000
	30% Engr & Cont	291,000
	Total	1,260,000

*July 1975 ENR 2248.

TABLE VIII-34

**CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 1 CAPACITY FROM 0.5 MGD
TO 0.7 MGD***

1)	Preliminary Treatment	26,000
2)	R.W. Pumps, Internal Piping	90,000
3)	Final Settling Tank (35′ ϕ, Dept = 8′)	74,000
4)	Chlorination Tank (16′ X 40′ X 6′)	47,000
5)	Misc. Electrical Instrumentation Heating and Vent.	58,000
6)	Yard Piping, Effluent Piping	68,000
	Subtotal	363,000
	30% Engr. & Cont.	109,000
	Total	472,000

*July 1975 ENR 2248.

TABLE VIII-35

CAPITAL COST BREAKDOWN FOR UPGRADING THE VILLAGE OF OWEGO STP SECONDARY TREATMENT CAPACITY TO 1 MGD*

1)	Secondary Pump Station (Max Flow = 3.0)	200,000
2)	Trickling Filter A = 4,370 SF 75'φ X 5'-6" Stone Depth	87,000
3)	Final Settling TKS A = 1,904 SF 2 - 14' X 68' X 7' SWD	78,000
4)	Chlorine Tanks Vol = 3,800 16' X 40' X 6' SWD	25,000
5)	Sludge Holding Tanks 2 - 19' X 22' X 9' SWD With Mixers	60,000
6)	Vacuum Filter Area = 100 SF including Building 8'φ X 4' Face	160,000
7)	Preliminary Treatment – Screens, Grit removal Conninutor, etc.	65,000
8)	Site Work – Added Fill, Yard Piping, Grading, etc.	110,000
	Subtotal	785,000
	30% Engr. & Cont.	235,000
	Total	1,020,000

*July 1975 ENR 2248.

TABLE VIII-36

CAPITAL COST BREAKDOWN FOR SECONDARY TREATMENT PLANT AT CHENANGO VALLEY (Capacity of 1.0 mgd)*

1.	Preliminary Treatment		$ 74,000
2.	Primary Settling Tanks (A = 1250 SF 2 - 12' x 52' x 8' SWD)		74,000
3.	Aeration Tanks (V = 33,700 CF 2 - 19' x 74' x 12' SWD)		85,000
4.	Diffused Aeration System		139,000
5.	Secondary Settling Tanks (A = 1430 SF 2 - 13 x 55 x 10' SWD)		79,000
6.	Sludge Digestion (V - 40,000 CF 2 - 35ϕ)		262,000
7.	Chlorine Contact Tanks (V = 3480 CF 2 - 8' x 31' x 7' SWD)		24,000
8.	Chlorine Feed Equipment		28,000
9.	Yardwork		120,000
10.	Outfall to Chenango River		115,000
		Subtotal	$1,000,000
		Engr. & Cont. @ 30%	300,000
		Total	$1,300,000

*July 1975 ENR 2248

TABLE VIII-36 (Continued)

(Expansion of STP Capacity from 1.0 mgd to 2.2 mgd)*

1.	Preliminary Treatment	$ 88,000
2.	Primary Settling Tanks (A = 1560 SF 2 - 15' x 52' x 8' SWD)	82,000
3.	Aeration Tanks (V = 40,800 SF 2 - 23' x 74' x 12' SWD)	99,000
4.	Diffused Aeration System	177,000
5.	Secondary Settling Tanks (A - 1760 SF 2 - 16' x 55' x 10' SWD)	87,000
6.	Sludge Digestion (V = 44,000 CF 1 - 54'ϕ)	277,000
7.	Chlorine Contact Tanks (V = 4340 CF 2 - 10' x 31' x 7' SWD)	27,000
8.	Chlorine Feed Equipment	31,000
9.	Yardwork	55,000
	Subtotal	$923,000
	Engr. & Cont. @ 30%	277,000
	TOTAL	$1,200,000

*July 1975 ENR 2248

TABLE VIII-37

CAPITAL COST BREAKDOWN FOR THE NANTICOKE INTERCEPTOR (AVERAGE DESIGN FLOW = 0.8 M.G.D.* PEAK DESIGN FLOW = 2.3 M.G.D.)

1) 18" sewer – 25,000' @ $38/LF	957,000
2) 110 manholes @ $1,100	120,000
3) Misc. items	33,000
4) Railroad crossing 27" casing 100' @ $270	27,000
5) Creek Crossing 200/LF @ $350/LF	70,000
Subtotal	1,207,000
30% Engr. & Contr.	362,000
Total	1,569,000

*use 18"ϕ sewer @ $S = 0.0012$ min. assume avg. depth = 12-14 feet.

July 1975 ENR 2248.

TABLE VIII-38

PLAN 2C
OPERATION AND MAINTENANCE COSTS

	First Year	Last Year	Description	Annual Cost ($ Million/Yr)	Present Worth[1] ($ Million)
Binghamton-Johnson City	1977	2026	Secondary	0.72	
	1977	2026	Storm Overflow	0.05	
					11.9
Endicott	1977	1982	Secondary	0.31	
	1983	2026	Secondary	0.36	
					5.1
Chenango	1977	1981	Secondary	0.086	
	1982	2026	Secondary	0.15	
					2.1
East Owego	1977	1991	Secondary	0.14	
	1992	2026	Secondary	0.19	
					2.4
West Owego	1977	1999	Secondary	0.04	
	2000	2026	Secondary	0.05	
					0.6
Owego Village	1977	2026	Microscreening	0.002	
	1977	2026	Secondary	0.07	
					1.1
				Total Present Worth:	23.2

[1] 50 years @ 6-1/8%.

TABLE VIII-39

PLAN 2C

CAPITAL COSTS (MILLION $) AND OPERATION AND MAINTENANCE COSTS ($$10^3$/Yr)

	Binghamton Johnson		Chenango		Endicott		E. Owego		W. Owego		Owego Village	
Year	Capital	O&M	Capital	O&M	Capital	O&M	Capital	O&M	Capital	O&M	Capital	O&M
1977	4.55	770	1.3	86	1.57	310		140		40	2.0	70
1982			1.2	150								
1983					1.95	360						
1985												
1989												
1990												
1991	.71											
1992							1.26	190				
1995												
2000									.47	50		
2010												
2018												
2026		770		150		360		190		50		70
Present Worth Million $	4.9	11.9	2.2	2.1	2.9	5.1	.5	2.4	.1	.6	2.0	1.1
Present Worth of Replacement Million $	6.1		.5		3.8		.8		.2		.5	
Total Present Worth Million $	22.9		4.8		11.8		3.7		0.9		3.6	

TABLE VIII-40

PLAN 2C COSTS – PRESENT WORTH[1]
(MILLION $'s)

	Chenango Valley	Binghamton-Johnson City	Endicott	East Owego	West Owego	Owego Village	Total
Wastewater Treatment							
Capital	1.7	1.1	1.3	0.5	0.1	1.0	5.7
O & M	1.5	7.3	4.4	1.9	0.4	0.7	16.2
Replacement	0.4	4.9	3.6	0.7	0.1	0.3	10.0
Total	3.6	13.3	9.3	3.1	0.6	2.0	31.9
Interceptors							
Capital	–	–	1.6	–	–	–	1.6
O & M	–	–	–	–	–	–	–
Replacement	–	–	–	–	–	–	–
Total	–	–	1.6	–	–	–	1.6
Storm Overflows							
Capital	–	3.6	–	–	–	1.0	4.6
O & M	–	0.8	–	–	–	0.1	0.9
Replacement	–	0.9	–	–	–	0.1	1.0
Total	–	5.3	–	–	–	1.2	6.5
Infiltration Control							
Capital	–	0.2	–	–	–	–	0.2
O & M	–	–	–	–	–	–	–
Replacement	–	–	–	–	–	–	–
Total	–	0.2	–	–	–	–	0.2
Sludge							
Capital	0.465	0.05	0.01	0.005	0.001	0.002	0.5
O & M	0.6	3.8	0.7	0.5	0.2	0.3	6.1
Replacement	0.1	0.3	0.2	0.1	0.1	0.1	0.9
Total	1.2	4.1	0.9	0.6	0.3	0.4	7.5
Totals							
Capital	2.2	4.9	2.9	0.5	0.1	2.0	12.6
O & M	2.1	11.9	5.1	2.4	0.6	1.1	23.2
Replacement	0.5	6.1	3.8	0.8	0.2	0.5	11.9
Total Plan Cost	4.8	22.9	11.8	3.7	0.9	3.6	47.7

[1] Based on a 50 year economic life @ 6-1/8% and ENR = 2248.

PLAN 3A

The intent of this plan is to maintain minimum daily average DO levels above 5 mg/l in the Susquehanna River during low flow conditions throughout the planning period and minimum instantaneous DO levels above 4 mg/l during storm overflow conditions. Plans 3A, 3B, and 3C are similar except for the degree of regionalization. The only additional requirement to maintain minimum daily average DO levels above 5 mg/l DO in the Susquehanna River (Plan 3) over the 4 mg/l system (Plan 2) is the provision of nitrification facilities for ammonia removal at the Binghamton-Johnson City STP during the 1990's. Other than the modification in capital improvements and manpower requirements (see Institutional Analysis Appendix), the detailed refinement of Plan 3A is virtually identical to Plan 2A.

MUNICIPAL WASTEWATER MANAGEMENT

Regionalization of STP's

The Broome County plants would be at the existing Binghamton-Johnson City and Endicott STP's. The wastewater generated from the entire Chenango Valley service area would be treated at the Binghamton-Johnson City STP. Tioga County plants would be at the Town of Owego STP's No. 1 (West Owego) and No. 2 (East Owego) and the Village of Owego STP.

Service areas and interceptors included in this plan are shown in Plate 4. The only additional interceptor to those included in the 1977 Baseline Plan connects the Chenango Valley service area to the north end of the City of Binghamton wastewater collection system.

Treatment Levels and Processes

Nitrification facilities would be added to the existing activated sludge secondary treatment process at the Binghamton-Johnson City STP by 1994. Secondary treatment would be sufficient at the other four STP's. The Owego Village treatment plant was assumed to be upgraded to provide secondary treatment using a trickling filter process (recently changed to a proposal for an activated sludge process). The activated sludge process at the Town of Owego STP No. 2 and the

trickling filter processes at the Village of Endicott and the Town of Owego STP No. 1 would be continued in use with their capacities expanded as the wastewater flow increases.

Infiltration Control Level in the City of Binghamton

For the level of treatment (secondary + nitrification) and the regionalization scheme assumed in this plan, an infiltration flow reduction of 3 mgd was found to be economically justifiable.

Nonstructural Measures for Flow Reduction

In this plan, the Village of Endicott and the Town of Owego would institute metered user rates. The increase in price the consumer would be paying should result in a reduction of water consumption, and, therefore, wastewater flows. An educational program to encourage the use of water saving devices could result in an estimated 20 percent reduction in the projected incremental increase in per capita flows. Although such a program could be undertaken, no formal educational activities have been specified in this plan.

STORM OVERFLOW MANAGEMENT

Microscreening units followed by chlorination would be provided near five overflow sites in the City of Binghamton (see Figure VIII-27). This system would maintain a minimum instantaneous DO above 4 mg/l during most storm conditions. A detailed description of the overflow treatment systems is given in Chapter VII. An estimate was also made of the costs of inflow control and microscreening treatment for the Village of Owego's combined sewer system.

SLUDGE MANAGEMENT

Several sludge management techniques have been analyzed in Chapter VI. The land application of liquid sludge is recommended, with landfill of dewatered sludge as a backup.

PERFORMANCE

Impact on Susquehanna River dissolved oxygen levels:

1. The minimum daily average dissolved oxygen level would be between 5 and 6 mg/l during low river flow and dry wastewater flow conditions.

2. The minimum instantaneous dissolved oxygen level would be between 4 and 5 mg/l during design storm conditions.

The pollutant mass loads discharged to the Susquehanna and Chenango Rivers by the STP's proposed in this alternative are summarized in Table VIII-41.

CONSTRUCTION SCHEDULE

Expansion of the existing wastewater facilities would be required during the planning period to handle the increasing flow expected (see Figures VIII-28 and VIII-29).

At the Binghamton-Johnson City STP, the raw wastewaters pumping capacity will be increased from 18.3 mgd to 29 mgd by the year 1977. Also, two aeration and clarifier units will be added to increase the capacity from 18.3 mgd to 26.9 mgd by 1977.

In addition to the secondary treatment expansions, nitrification will be required in the year 1994 to maintain river minimum daily average DO above 5.0 mg/l at critical conditions. Suspended growth nitrification, designed in detail in Chapter V, will be used with capacities to last until the end of the planning period. The required expansions are shown on Figure VIII-30.

The Chenango service area wastewater will be transmitted to the City of Binghamton wastewater collection system through a 16" force main that has a design flow of 2.2 mgd. A 1.38 million gallon holding tank is used to equalize the wastewater flow and hence reduce the pump station and the force main costs. This interceptor will be built by 1977.

Storm overflow control facilities are assumed to be constructed at Binghamton by 1977. Five microscreening facilities having a total treatment capacity of 39.5 mgd will be located as shown in Figure VIII-27.

At Endicott STP, the secondary treatment units will need expansion from 7.7 mgd to 9.2 mgd capacity by 1983. The trickling filters, secondary clarifitions, digesters, chlorination tank, raw wastewater and sludge pumping capacities will be expanded (see Figure VIII-31).

The Nantocike service area wastewater will be transmitted to the Endicott STP through an 18 inch diameter sewer that has a design average flow of 0.8 mgd and a peak flow of 2.3 mgd. The sewer length is 25,000 feet. This interceptor will be built by 1977.

At the Town of Owego STP No. 2, the plant treatment capacity will be upgraded from the present 2.0 mgd to 3.0 mgd by 1992. The additional units include preliminary treatment; a primary settling tank with a 1,250 square foot capacity; a 31,000 cubic foot aeration tank and air system; a sludge thickener for 4 mgd; a digester 45' by 20'; a chlorination tank 25' by 18' and 8' deep; and two final clarifiers having 3 mgd capacity and each is 55' by 12' and 8.5' deep (see Figure VIII-32).

At the Town of Owego STP No. 1, the plant capacity will be expanded by the year 2000 from the present 0.5 mgd to 0.7 mgd. The added facilities will include a preliminary treatment capacity, a final settling tank, and a chlorination tank (see Figure VIII-33).

At the Village of Owego, the treatment facilities will be upgraded to provide secondary treatment by 1977. The capacity of the upgraded facility will be 1 mgd. The facilities added were assumed to include a trickling filter with a surface area of 4,370 square feet, 75' diameter and 5'6" in depth; a secondary pump station designed to handle a maximum flow of 3.0 mgd; two final settling tanks each 14' by 68' and 7' deep; a chlorine tank, volume 3,800 cubic feet; two sludge holding tanks with mixers, each 19' by 22' and 9' deep; and a vacuum filter with an area of 100 square feet (see Figure VIII-34).

At the Village of Owego, a microscreening treatment unit with a capacity of 3 mgd will be built by 1977 to provide treatment of storm overflows. Also, steps will be taken to control direct stream infiltration into the sewer system.

A summary of the construction schedule of wastewater treatment facilities needed at each service area is given in Table VIII-42.

COST ANALYSIS

The capital costs breakdown for the expansion and/or upgrading of the existing treatment facilities at Binghamton-Johnson City, the Village of Endicott, the Town of Owego STP No. 1, the Town of Owego STP No. 2, and the Village of Owego are shown in Tables VIII-43 through VIII-47.

The capital costs breakdown for the Chenango interceptor and the Nanticoke interceptor are shown in Tables VIII-48 and VIII-49, respectively.

The detailed operation and maintenance costs of the waste-water treatment plants for each service area during the planning period are given in Table VIII-50.

The capital, O&M, and replacement expenditures for each service area are summarized in Table VIII-51. Also shown in that table are the years these expenditures will be incurred which correspond to the construction schedule described in the previous section. The replacement costs were estimated using an equivalent annual sinking fund approach.

The present worth of all the expenditures during the 50-year planning period is given in Table VIII-52 for each service area. The present worth costs for this plan are: capital -- \$15.8 million, O&M -- \$22.9 million, and replacement -- \$11.8 million, for a total cost of \$50.5 million.

CITY OF BINGHAMTON
LOCATION OF PLANNED OVERFLOW CONTROL FACILITIES

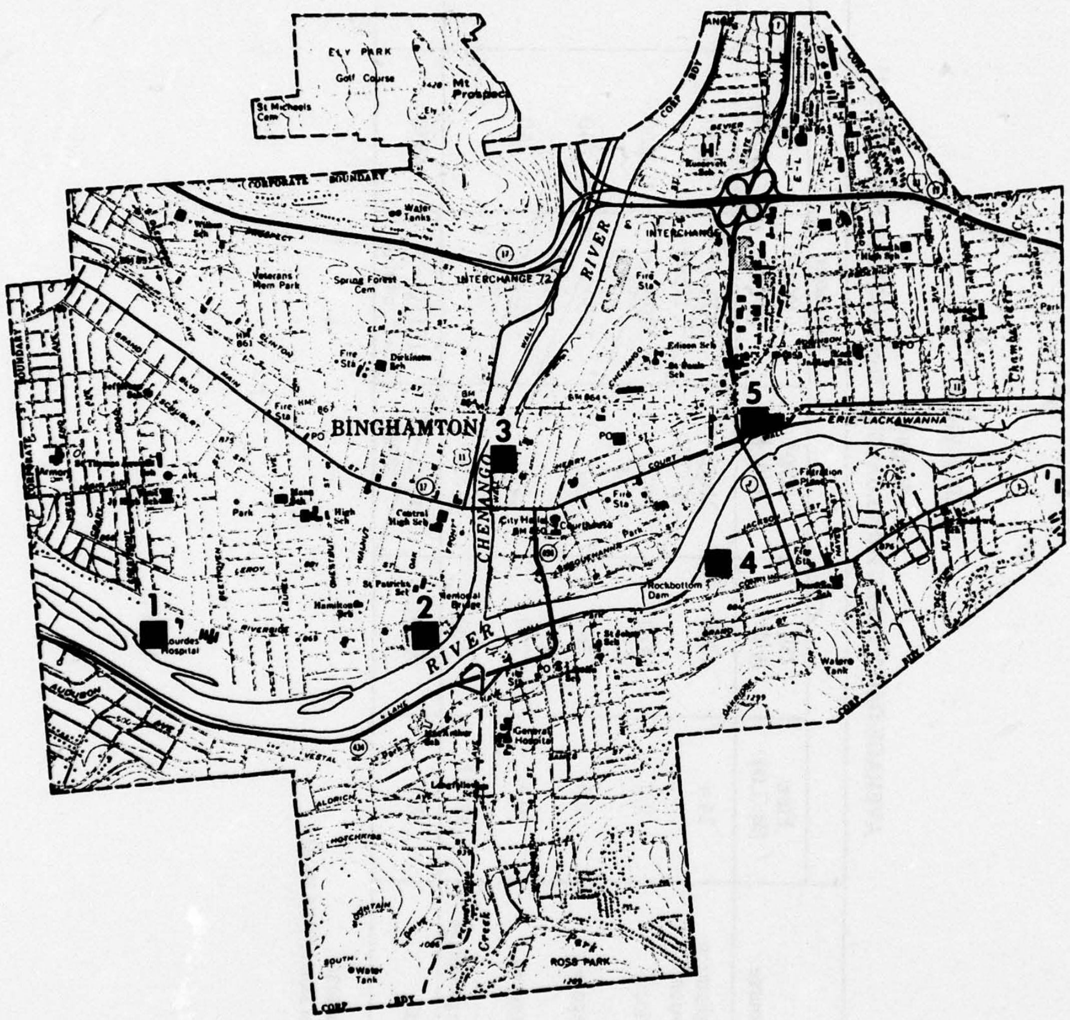


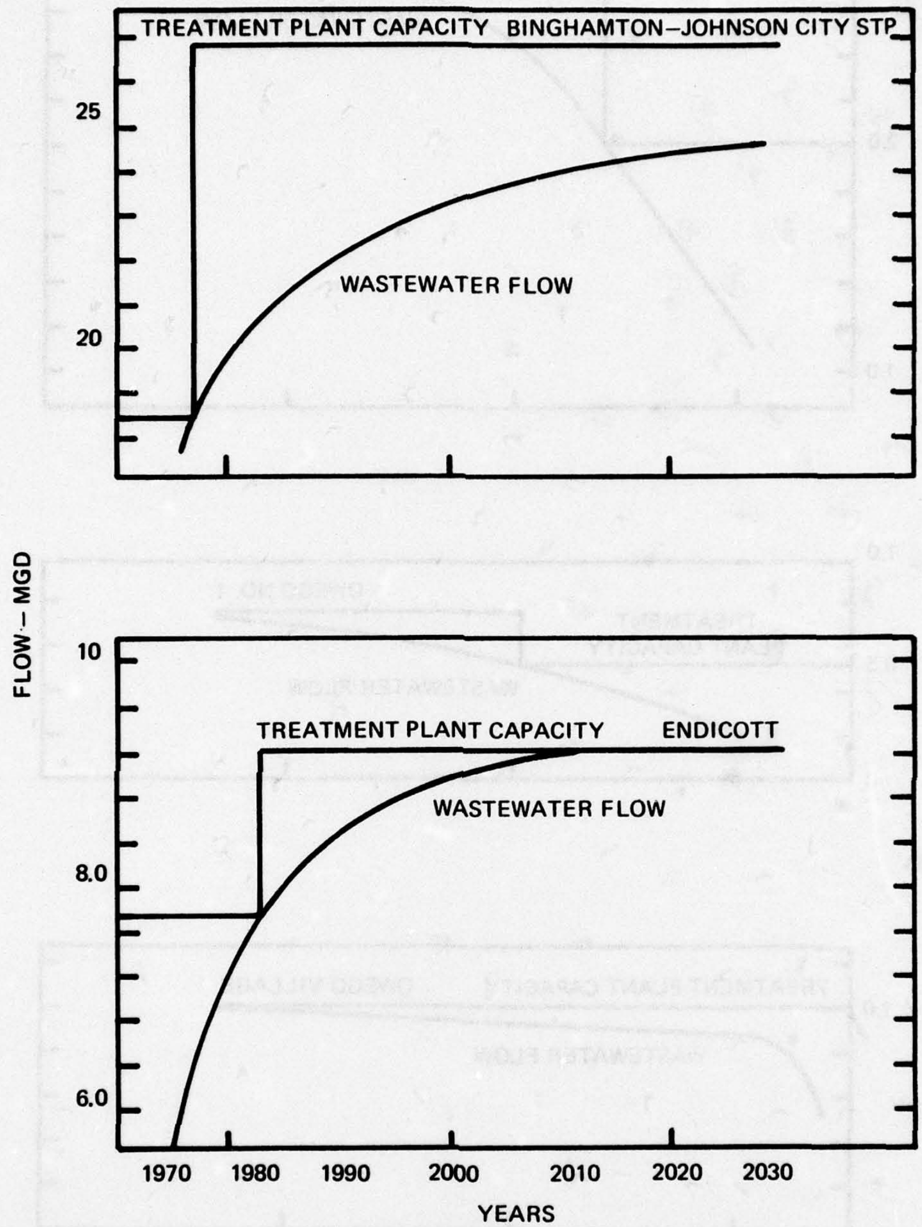
FIGURE VIII-27

TABLE VIII-41

PLAN 3A
AVERAGE DAILY POLLUTION LOADS TO THE SUSQUEHANNNA RIVER[1]

Source	Parameter					
	Flow (M.G.D)	SS lb/day	BOD[2] lb/day	NOD[2] lb/day	N lb/day	P lb/day
Binghamton-Johnson	24.4	4200	2100/3700	5500/11000	3480	870
Endicott	9.2	1470	1470	5200/8800	2260	600
Owego #2	2.8	360	360	1200/2500	640	150
Owego #1	0.7	200	200	1250/2100	450	120
Owego Village	0.97	100	100			
Total	38.1	6330	4230/5830	13150/24400	6830	1740

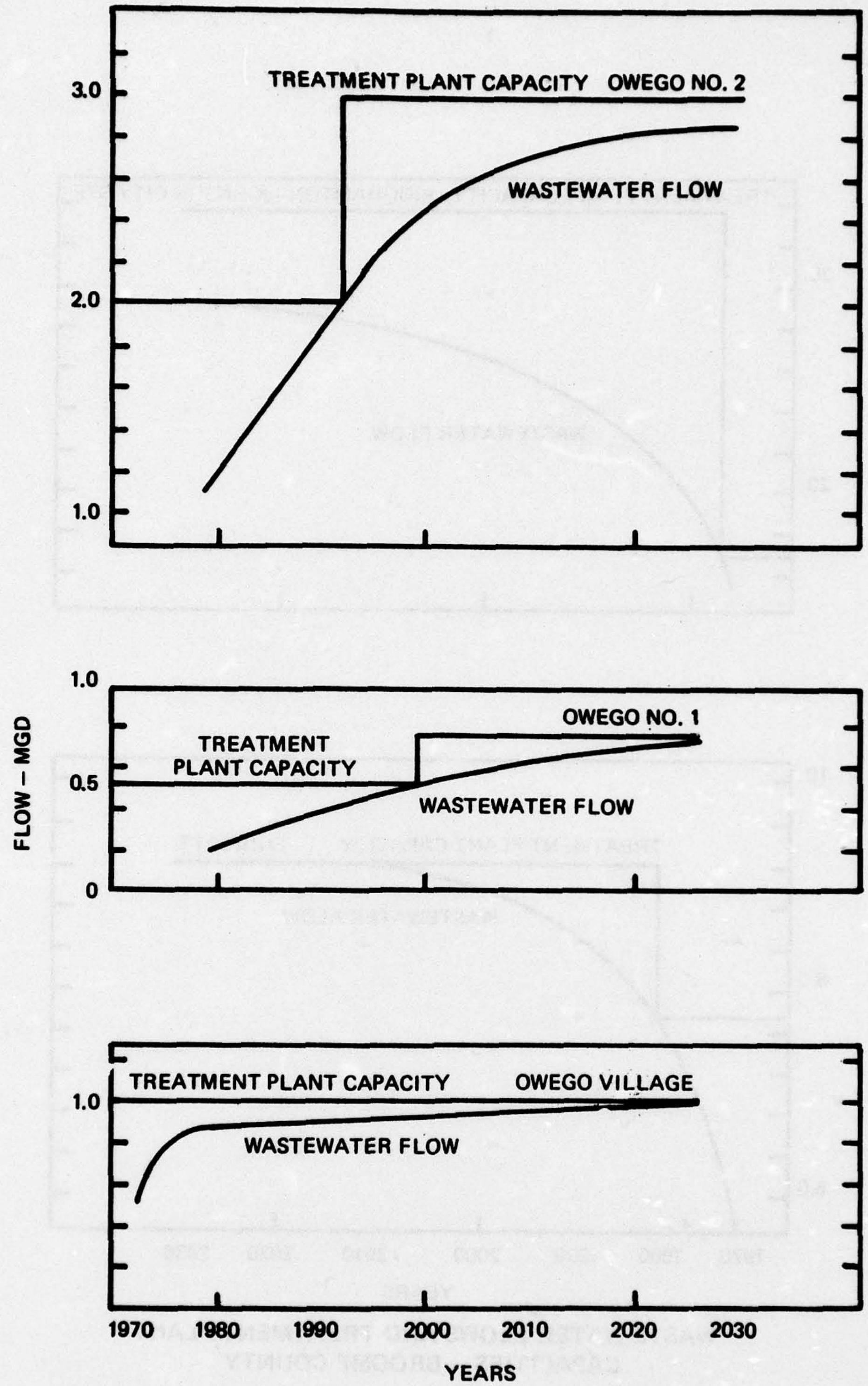
[1]Year 2020
[2]Warm Months/Cold Months



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – BROOME COUNTY

PLAN 3A

FIGURE VIII-23



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – TIOGA COUNTY

FIGURE VIII-29

PLANS 2A, 2B, 2C, 3A, 3B, and 3C

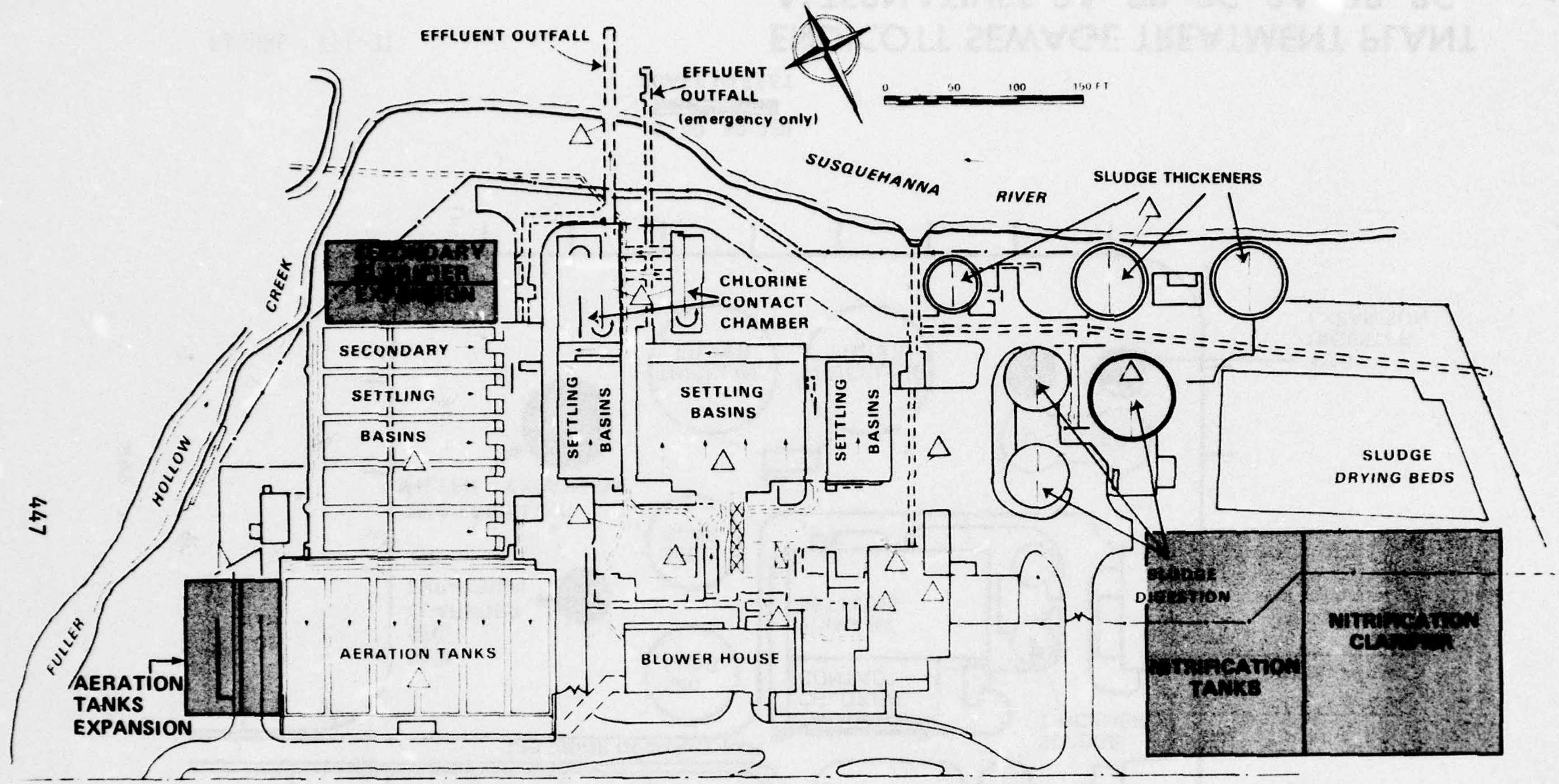


FIGURE VIII-30

ALTERNATIVES 3A, 3B, 3C

BINGHAMTON-JOHNSON CITY STP

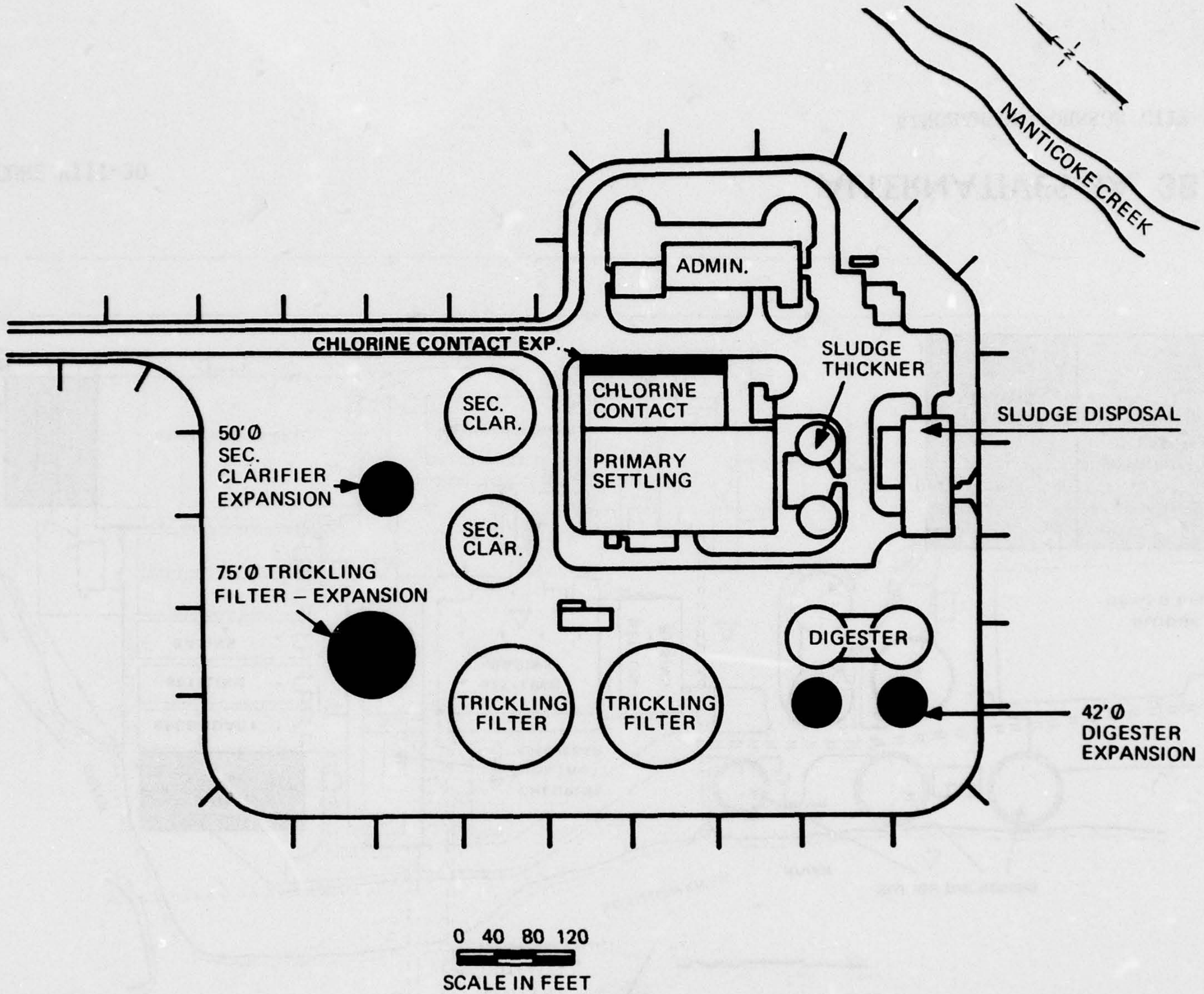
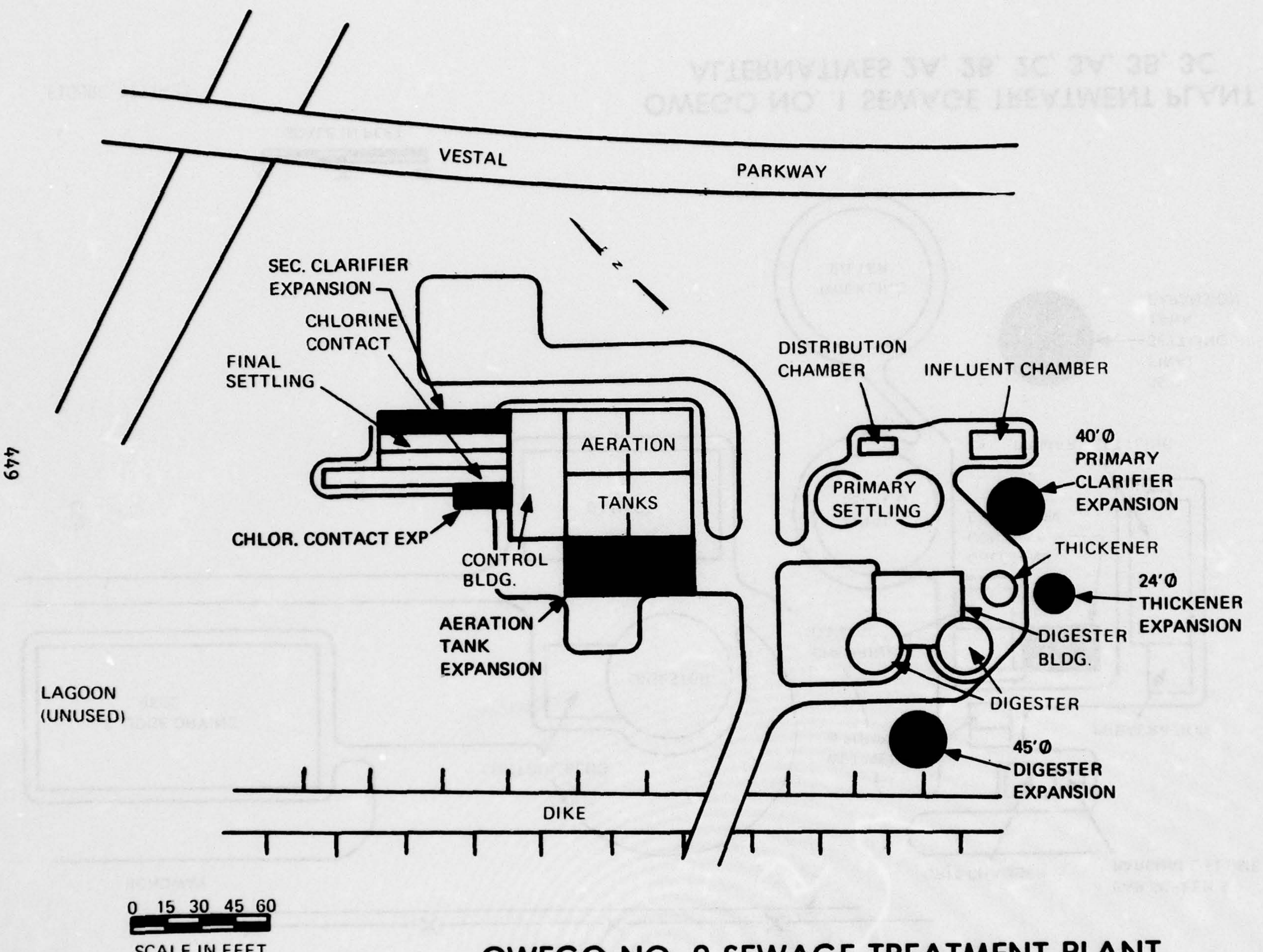


FIGURE VIII-31

ENDICOTT SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C



OWEGO NO. 2 SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-32

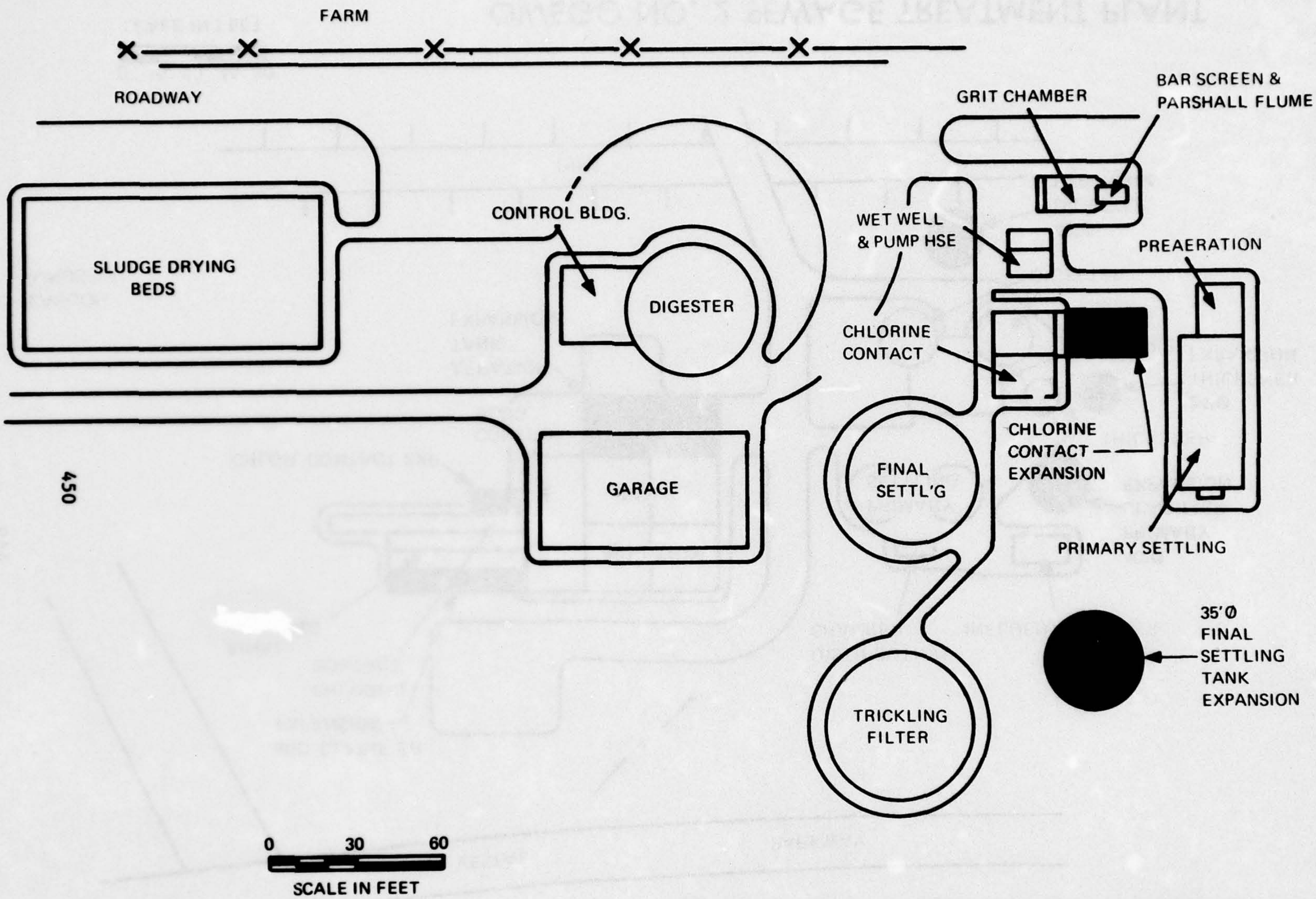
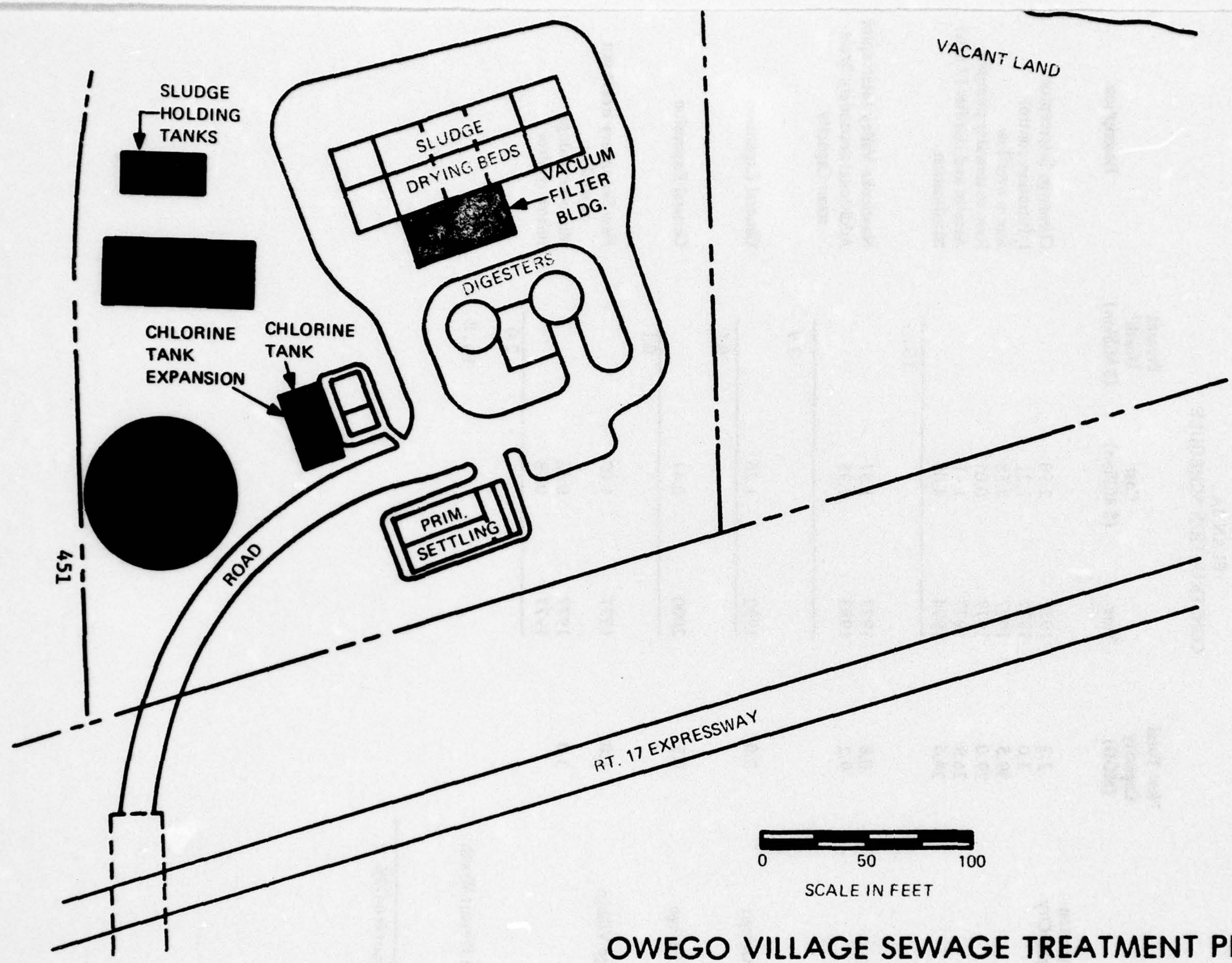


FIGURE VIII-33

OWEGO NO. 1 SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C



OWEGO VILLAGE SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-34

TABLE VIII-42

PLAN 3A
CONSTRUCTION SCHEDULE

	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[1] ($ Million)	Description
Binghamton-Johnson City	2.2	1977	2.50		Chenango Interceptor
	3.0	1977	1.21		Infiltration control
	39.5	1977	3.58		Storm overflow
	29.0	1977	0.05		Raw wastewater pumping
	26.9	1977	1.41		Aerator and clarifier (2 sets)
	24.4	1994	4.05		Nitrification
				10.3	
Endicott	0.8	1977	1.57		Nanticoke Valley Interceptor
	9.2	1983	1.95		Additional Secondary Treatment Capacity
				2.9	
East Owego	3.0	1992	1.26		General Expansion
				0.5	
West Owego	0.7	2000	0.47		General Expansion
				0.1	
Owego Village	1.0	1977	1.02		New Secondary treatment capability
	3.0	1977	0.42		Micro-screening
		1977	0.58		Inflow control
				2.0	
Total Present Worth				15.8	

[1] 50 years @ 6-1/8%

TABLE VIII-43

CAPITAL COST BREAKDOWN FOR AN AERATION TANK AND SECONDARY CLARIFIER SET AT THE BINGHAMTON–JOHNSON CITY STP*

1)	Clarifier	$190,000
2)	Aeration Tank (30′ × 100′)	225,000
3)	Yard Piping & Renovation Instrumentation	80,000
4)	Additional Pumping and Valves	50,000
	Sub Total	545,000
	Eng. & Cont. @ 30%	164,000
	Total	$709,000

*July 1975 ENR 2248

TABLE VIII-44

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE SECONDARY TREATMENT
FACILITIES AT ENDICOTT STP FROM 7.7
TO 9.2 MGD CAPACITY*

1)	Preliminary Treatment		44,000
2)	Raw Wastes Pumps		74,000
3)	Trickling Filter (75'ϕ - 6'deep)		105,000
4)	Secondary Clarifier (50'ϕ X B'- 6 SWD)		158,000
5)	Chlorine Tank & Feed		51,000
6)	Digester Incl Bld Co. (42'ϕ - 30"high)		440,000
7)	Sludge Pumping		84,000
8)	Piles (for all structures)		345,000
9)	Misc. Electricity, Heating and Ventilation		148,000
10)	Yard Piping		54,000
		Subtotal	1,503,000
		Engineering & Contingencies @ 30%	451,000
		Total	1,954,000

*July 1975 ENR 2248.

TABLE VIII-45

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 2 CAPACITY FROM 2.0 MGD
TO 3 MGD*

1)	Prelimination Treatment & R.W. Pumps, wet well	137,000
2)	Preliminary Settling Tanks 1 tank – 1250 SF	63,000
3)	Aeration Tank & Air System 31,000 CF	116,000
4)	Pumps – Recirculating Piping, Structure Upgrade	80,000
5)	Sludge Thickener (for 4 MGD 24′ Diameter)	42,000
6)	Digester 45′ϕ X 20′ Diameter	220,000
7)	Chlorine Tank & Feed System	58,000
8)	Sludge Pumping	47,000
9)	Final Clarifiers 55′ X 12′ X 8.5′ depth	74,000
10)	Misc. – Electricity, Heating and Ventilation Sitework	132,000
	Subtotal	969,000
	30% Engr & Cont	291,000
	Total	1,260,000

*July 1975 ENR 2248.

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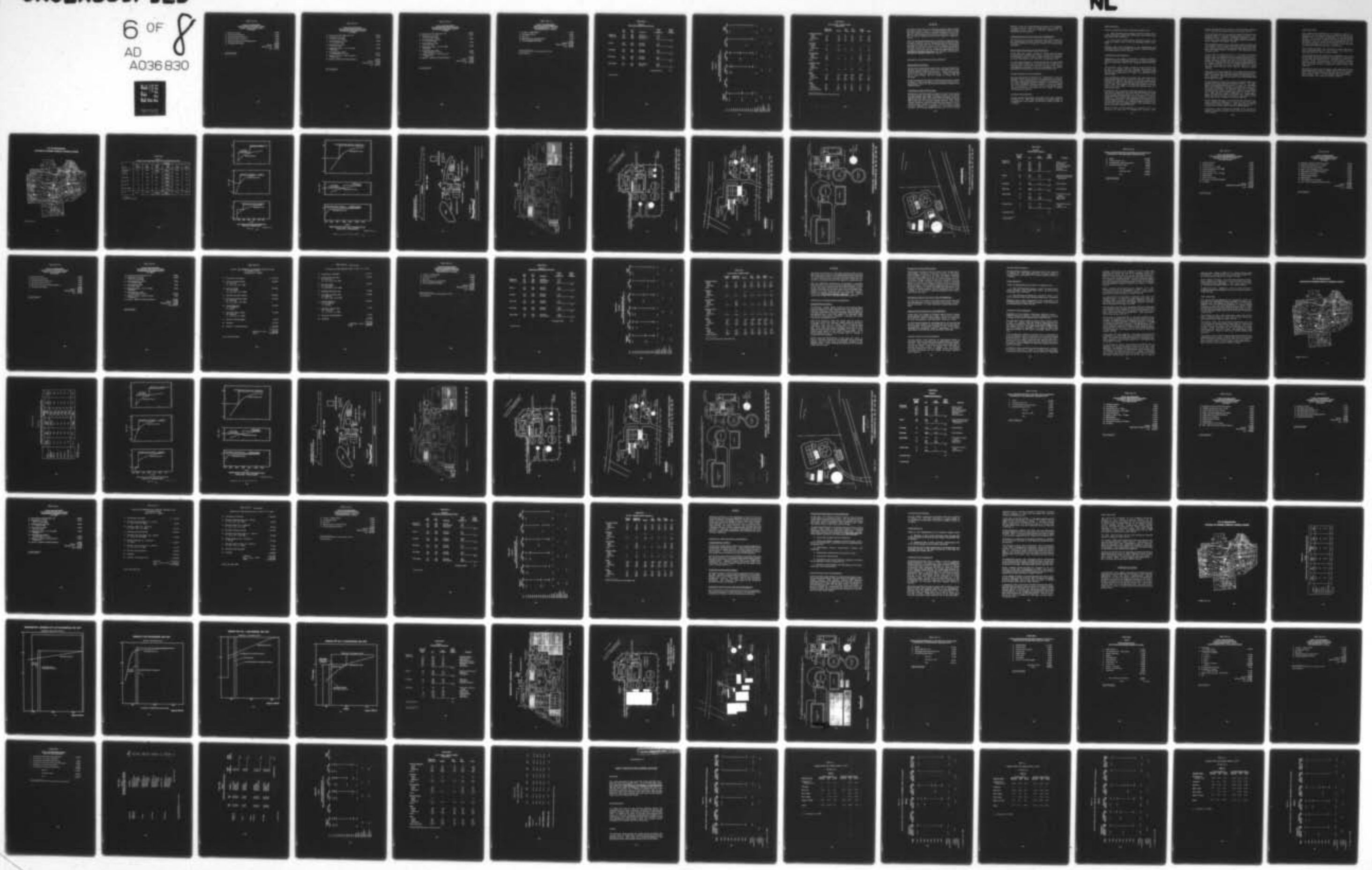


TABLE VIII-46

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 1 CAPACITY FROM 0.5 MGD
TO 0.7 MGD*

1)	Preliminary Treatment	26,000
2)	R.W. Pumps, Internal Piping	90,000
3)	Final Settling Tank (35′ϕ, Dept = 8′)	74,000
4)	Chlorination Tank (16′ X 40′ X 6′)	47,000
5)	Misc. Electrical Instrumentation Heating and Vent.	58,000
6)	Yard Piping, Effluent Piping	68,000
	Subtotal	363,000
	30% Engr. & Cont.	109,000
	Total	472,000

*July 1975 ENR 2248.

TABLE VIII-47

CAPITAL COST BREAKDOWN FOR UPGRADING THE VILLAGE OF OWEGO STP SECONDARY TREATMENT CAPACITY TO 1 MGD*

1)	Secondary Pump Station (Max Flow = 3.0)	200,000
2)	Trickling Filter A = 4,370 SF 75'ϕ X 5'-6" Stone Depth	87,000
3)	Final Settling TKS A = 1,904 SF 2 - 14' X 68' X 7' SWD	78,000
4)	Chlorine Tanks Vol = 3,800 16' X 40' X 6' SWD	25,000
5)	Sludge Holding Tanks 2 - 19' X 22' X 9' SWD With Mixers	60,000
6)	Vacuum Filter Area = 100 SF including Building 8'ϕ X 4' Face	160,000
7)	Preliminary Treatment – Screens, Grit removal Conninutor, etc.	65,000
8)	Site Work – Added Fill, Yard Piping, Grading, etc.	110,000
	Subtotal	785,000
	30% Engr. & Cont.	235,000
	Total	1,020,000

*July 1975 ENR 2248.

TABLE VIII-47

CAPITAL COST BREAKDOWN FOR UPGRADING THE VILLAGE OF OWEGO STP SECONDARY TREATMENT CAPACITY TO 1 MGD*

1)	Secondary Pump Station (Max Flow = 3.0)	200,000
2)	Trickling Filter A = 4,370 SF 75′ϕ X 5′-6″ Stone Depth	87,000
3)	Final Settling TKS A = 1,904 SF 2 - 14′ X 68′ X 7′SWD	78,000
4)	Chlorine Tanks Vol = 3,800 16′ X 40′ X 6′ SWD	25,000
5)	Sludge Holding Tanks 2 - 19′X 22′X 9′SWD With Mixers	60,000
6)	Vacuum Filter Area = 100 SF including Building 8′ϕ X 4′ Face	160,000
7)	Preliminary Treatment – Screens, Grit removal *Conninutor, etc.*	65,000
8)	Site Work – Added Fill, Yard Piping, Grading, etc.	110,000
	Subtotal	785,000
	30% Engr. & Cont.	235,000
	Total	1,020,000

*July 1975 ENR 2248.

TABLE VIII-49

CAPITAL COST BREAKDOWN
FOR THE NANTICOKE INTERCEPTOR
(AVERAGE DESIGN FLOW = 0.8 M.G.D.*
PEAK DESIGN FLOW = 2.3 M.G.D.)

1)	18" sewer – 25,000' @ $38/LF	957,000
2)	110 manholes @ $1,100	120,000
3)	Misc. items	33,000
4)	Railroad crossing 27" casing 100' @ $270	27,000
5)	Creek Crossing 200/LF @ $350/LF	70,000
	Subtotal	1,207,000
	30% Engr. & Contr.	362,000
	Total	1,569,000

*use 18" ϕ sewer @ S = 0.0012 min. assume avg. depth = 12-14 feet.

July 1975 ENR 2248.

TABLE VIII-50

PLAN 3A
OPERATION AND MAINTENANCE COSTS

	First Year	Last Year	Description	Annual Cost ($ Million/Yr)	Present Worth[1] ($ Million)
Binghamton-Johnson City	1977	2026	Secondary	0.78	
	1977	2026	Storm Overflow	0.05	
	1994	2026	Nitrification	0.17	
					13.7
Endicott	1977	1982	Secondary	0.31	
	1983	2026	Secondary	0.36	
					5.1
East Owego	1977	1991	Secondary	0.14	
	1992	2026	Secondary	0.19	
					2.4
West Owego	1977	1999	Secondary	0.04	
	2000	2026	Secondary	0.05	
					0.6
Owego Village	1977	2026	Microscreening	0.002	
	1977	2026	Secondary	0.07	
					1.1
				Total Present Worth:	22.9

[1] 50 years @ 6-1/8%.

TABLE VIII-51

PLAN 3A

CAPITAL COSTS (MILLION \$) AND OPERATION AND MAINTENANCE COSTS (\$$10^3$/Yr)

Year	Binghamton Johnson Capital	Binghamton Johnson O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	E. Owego Capital	E. Owego O&M	W. Owego Capital	W. Owego O&M	Owego Village Capital	Owego Village O&M
1977	8.75	830			1.57	310		140		40	2.0	70
1983					1.95	360						
1985												
1989												
1990												
1992							1.26	190				
1994	4.05	1000										
1995												
2000									.47	50		
2010												
2018												
2026		1000				360		190		50		70
Present Worth Million \$	10.3	13.7			2.9	5.1	.5	2.4	.1	.6	2.0	1.1
Present Worth of Replacement Million \$	6.5				3.8		.8		.2		.5	
Total Present Worth Million \$	30.5				11.8		3.7		0.9		3.6	

TABLE VIII-52

PLAN 3A COSTS – PRESENT WORTH[1]
(MILLION $'S)

	Binghamton–Johnson City	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment						
Capital	2.9	1.3	0.5	0.1	1.0	5.8
O & M	8.6	4.4	1.9	0.4	0.7	16.0
Replacement	5.3	3.6	0.7	0.1	0.3	10.0
TOTAL	16.8	9.3	3.1	0.6	2.0	31.8
Interceptors						
Capital	2.5	1.6	–	–	–	4.1
O & M	–	–	–	–	–	–
Replacement	–	–	–	–	–	–
TOTAL	2.5	1.6	–	–	–	4.1
Storm Overflows						
Capital	3.6	–	–	–	1.0	4.6
O & M	0.8	–	–	–	0.1	0.9
Replacement	0.9	–	–	–	0.1	1.0
TOTAL	5.3	–	–	–	1.2	6.5
Infiltration Control						
Capital	1.2	–	–	–	–	1.2
O & M	–	–	–	–	–	–
Replacement	–	–	–	–	–	–
TOTAL	1.2	–	–	–	–	1.2
Sludge						
Capital	0.05	0.01	0.005	0.001	0.002	0.1
O & M	4.3	0.7	0.5	0.2	0.3	6.0
Replacement	0.3	0.2	0.1	0.1	0.1	0.8
TOTAL	4.7	0.9	0.6	0.3	0.4	6.9
Totals						
Capital	10.3	2.9	0.5	0.1	2.0	15.8
O & M	13.7	5.1	2.4	0.6	1.1	22.9
Replacement	6.5	3.8	0.8	0.2	0.5	11.8
TOTAL PLAN COST	30.5	11.8	3.7	0.9	3.6	50.5

[1] Based on a 50 year economic life @ 6 1/8% and ENR = 2248

PLAN 3B

The intent of this plan is to maintain minimum daily average DO levels above 5 mg/l in the Susquehanna River during low flow conditions and the minimum instantaneous DO level above 4 mg/l during storm overflow conditions. Plans 3A, 3B, and 3C are similar except for the degree of regionalization.

The only additional requirement to maintain a minimum daily average 5 mg/l DO in the Susquehanna River (Plan 3) over the 4 mg/l system (Plan 2) is the provision of nitrification facilities for ammonia removal at the Binghamton-Johnson City STP during the 1990's. Other than the modifications in capital improvements and manpower requirements (see Institutional Analysis Appendix), the detailed refinement of Plan 3B is virtually identical to Plan 2B.

MUNICIPAL WASTEWATER MANAGEMENT

Regionalization of STP's

The Broome County plants would be the existing Binghamton-Johnson City and Endicott STP's and a new STP to serve the entire Chenango Valley service area. Tioga County plants would be the Town of Owego STP's No. 1 (West Owego) and No. 2 (East Owego), and the Village of Owego STP (all existing).

Service areas and interceptors included in this plan are shown in Plate 5. With the exception of the Chenango Valley sewerage system, these are the same interceptors included in the Baseline Plan.

Treatment Levels and Processes

Nitrification facilites would be added by 1994 to the existing activated sludge secondary treatment process at the Binghamton-Johnson City STP. Secondary treatment would be sufficient at the other five STP's. The new Chenango Valley STP would use an activated sludge process. The Owego Village treatment plant was assumed to be upgraded to provide secondary treatment using a trickling filter process (recently changed to a proposal for an activated sludge process). The activated sludge process at the Town of Owego

STP No. 2 and the trickling filter processes at the Villagee of Endicott and the Town of Owego STP No. 1 would be continued in use, with their capacities upgraded as the wastewater flow increases.

Infiltration Control Level in the City of Binghamton

For the level of treatment (secondary plus nitrification) and the regionalization scheme assumed in this plan, an infiltration flow reduction of 3 mgd was found to be economically justifiable.

Nonstructural Measures for Flow Reduction

In this plan, the Village of Endicott and the Town of Owego would institute metered user rates. The increase in price the consumer would be paying should result in a reduction of water consumption, and, therefore, wastewater flows.

An educational program to encourage the use of water saving devices could result in an estimated 20 percent reduction in the projected incremental increase in per capita flows. Although such a program could be undertaken, no formal educational activities have been specified in this plan.

STORM OVERFLOW MANAGEMENT

Microscreening devices followed by chlorination would be provided near five overflow sites in Binghamton (see Figure VIII-35). This would maintain a minimum instantaneous DO of 4 mg/l during most storm conditions. A detailed description of the overflow treatment system is given in Chapter VII. An estimate was also made of the cost of inflow control and microscreening treatment for the Village of Owego combined sewer system.

SLUDGE MANAGEMENT

Several sludge management techniques have been analyzed in Chapter VI. The land application of liquid sludge is recommended with landfill of dewatered sludge provided as a backup.

PERFORMANCE

Impact on Susquehanna River dissolved oxygen levels:

1. The minimum daily average dissolved oxygen level would be between 5 and 6 mg/l during low river flow and dry wastewater flow conditions.

2. The minimum instantaneous dissolved oxygen level would be between 4 and 5 mg/l during design storm conditions.

Pollutant mass loads discharged to the Susquehanna and Chenango Rivers by the STP's proposed in this alternative are summarized in Table VIII-53.

CONSTRUCTION SCHEDULE

Expansion of the existing wastewater facilities would be required during the planning period at all STP's to handle the increasing flow expected (see Figures VIII-36 and VIII-37).

The Chenango Valley sewage treatment plant would be built by 1977 to conform with the Federal requirements for secondary treatment (see Figure VIII-38). The plant would have an initial capacity of 1.7 mgd, to be expanded to 2.2 mgd by 1985.

At the Binghamton-Johnson City STP, the raw wastewaters pumping capacity will be increased from 18.3 mgd to 29 mgd by 1977. Also by this year, one aerator and clarifier unit will be added to increase the capacity from 18.3 mgd to 21.5 mgd, with another aeration and clarifier unit to be added by the year 2000, to increase the capacity from 21.5 mgd to 24.5 mgd.

In addition to the secondary treatment expansions, nitrification will be required in the year 1994 to maintain river DO minimum daily average above 5.0 mg/l at critical conditions. Suspended growth nitrification, designed in detail in Chapter V, will be used with capacities to last until the end of the planning period. The expansions are virtually identical to those for Plan 3-A, with the exception being that the nitrification units are slightly smaller. The additions to this plant are shown on Figure VIII-39.

Storm overflow control facilities are assumed to be constructed at the City of Binghamton by 1977. Five

microscreening facilities having a total treatment capacity of 39.5 mgd will be located as shown in Figure VIII-35.

At the Endicott STP, the secondary treatment units will need expansion from 7.7 mgd to 9.2 mgd capacity by 1983. The trickling fiilters, secondary clarifiers, digesters, chlorination tank, raw wastewater and sludge pumping capacities will be expanded (see Figure VIII-40).

The Nanticoke service area wastewater will be transmitted to the Endicott STP through an 18-inch diameter sewer that has a design average flow of 0.8 mgd and a peak flow of 2.3 mgd. The sewer length is 25,000 feet. This interceptor will be built by 1977.

At the Town of Owego STP No. 2, the plant treatment capacity will be upgraded from the present 2.0 mgd to 3.0 mgd by 1992. The additional units include preliminary treatment; a primary settling tank with a 1,250 SF capacity; a 31,000 CF aeration tank and air system; a sludge thickener for 4 mgd; a digester 45' by 20'; a chlorination tank 25' by 18' and 8' deep; and two final clarifiers having a 3 mgd capacity, each being 55' by 12' and 8.5' deep (see Figure VIII-41).

At the Town of Owego STP No. 1, the plant capacity will be expanded by the year 2000 from the present 0.5 mgd to 0.7 mgd. The added facilities will include a preliminary treatment capacity, a final settling tank, and a chlorination tank (see Figure VIII-42).

At the Village of Owego, the treatment facilities will be upgraded to provide secondary treatment by 1977. The capacity of the upgraded facility will be 1 mgd. The facilities added were assumed to include a trickling filter with a surface area of 4,370 SF, 75' diameter and 5'6" in depth; a secondary pump station designed to handle a maximum flow of 3.0 mgd; two final settling tanks each 14' by 68' and 7' deep, chlorine tank volume 3,800 CF, two sludge holding tanks with mixers each 19' by 22' and 9' deep, and a vacuum filter with an area of 100 SF (see Figure VIII-43).

At the Village of Owego, a microscreening treatment unit with a capacity of 3 mgd will be built by 1977 to provide treatment of storm overflows. Also steps will be taken to control direct stream inflow into the existing system.

A summary of the construction schedule of the wastewater treatment facilities needed at each service area is given in Table VIII-54.

COST ANALYSIS

The capital costs breakdown for the expansion and/or upgrading of the existing treatment facilities at Binghamton-Johnson City, the Village of Endicott, the Town of Owego STP No. 1, the Town of Owego STP No. 2, and the Village of Owego are shown in Tables VIII-55 through VIII-59, respectively. Also, in Tables VIII-60 and VIII-61 are the capital costs breakdown for the new Chenango Valley STP and the Nanticoke Valley interceptor, respectively.

The detailed operation and maintenance costs of the wastewater treatment plants for each service area during the planning period are given in Table VIII-62.

The capital, O&M and replacement expenditures for each service area are summarized in Table VIII-63. Also shown in that table are the years these expenditures will be incurred. These correspond to the construction schedule described in the previous section. The replacement costs were estimated using an equivalent annual sinking fund approach.

The present worth of all the expenditures during the 50-year planning period is given in Table VIII-64 for each service area. The present worth costs for this plan are: capital -- $14.8 million, O&M -- $24.1 million, and replacement -- $12.2 million, for a total cost of $51.1 million.

CITY OF BINGHAMTON

LOCATION OF PLANNED OVERFLOW CONTROL FACILITIES

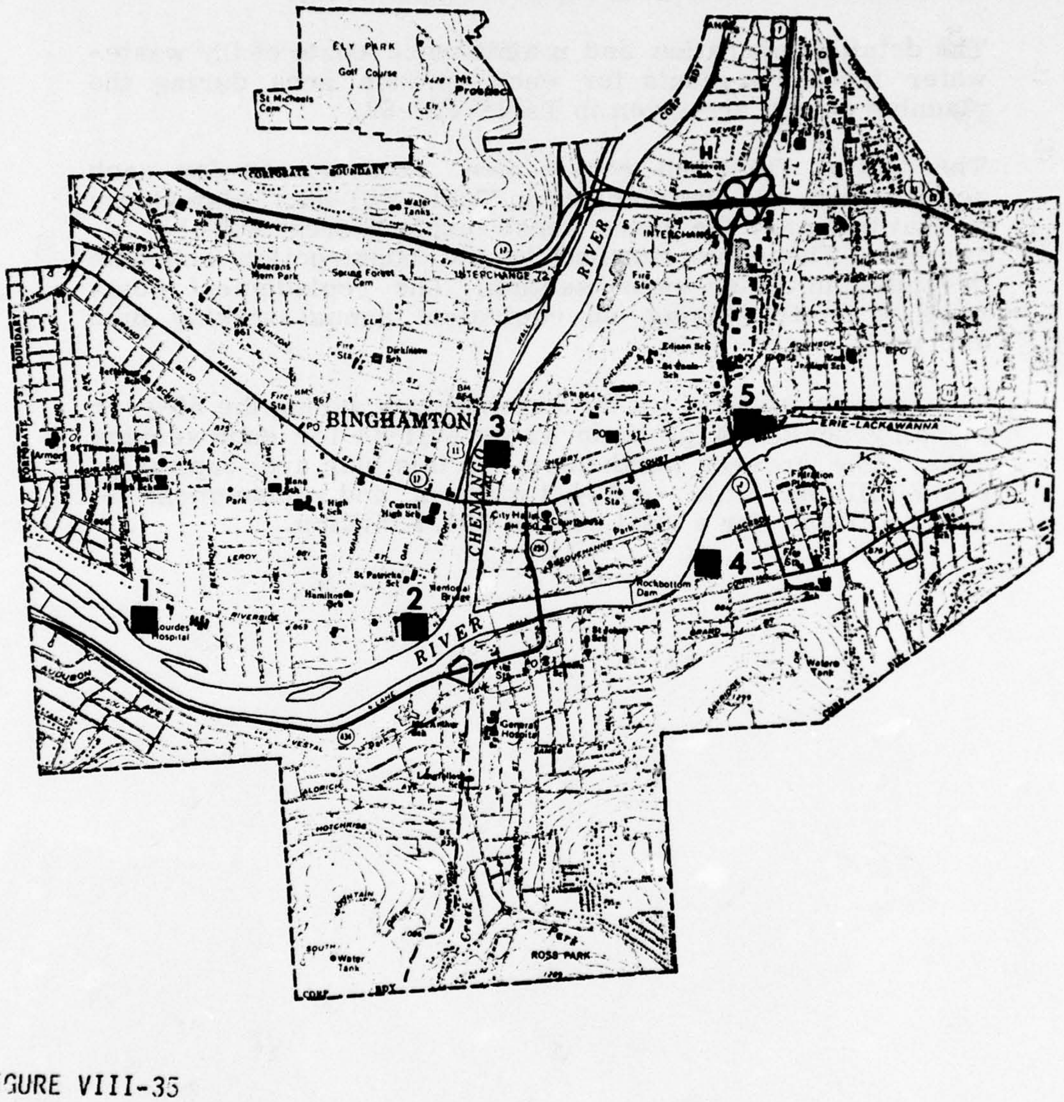


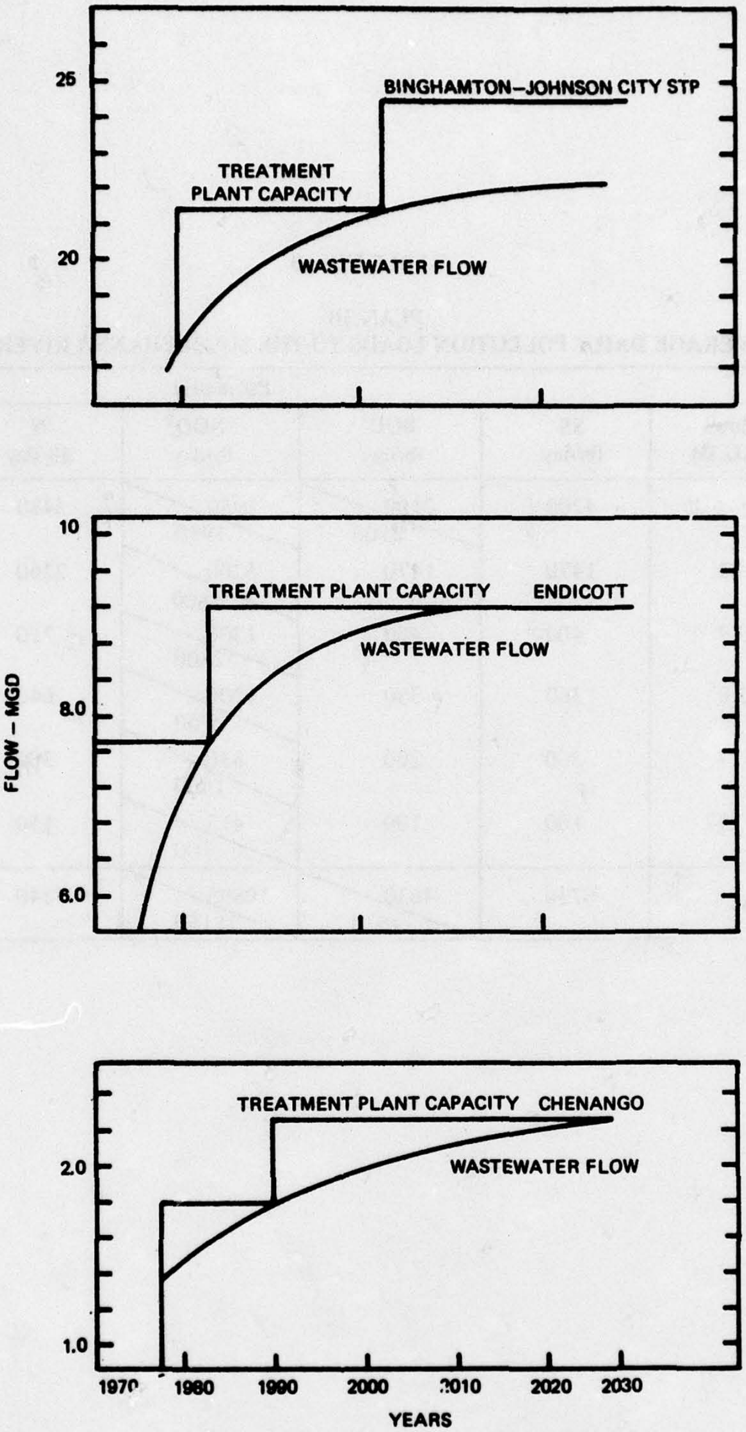
FIGURE VIII-35

TABLE VIII-53

PLAN 3B
AVERAGE DAILY POLLUTION LOADS TO THE SUSQUEHANNA RIVER[1]

	Parameter					
Source	**Flow (M.G.D)**	**SS lb/day**	**BOD[2] lb/day**	**NOD[2] lb/day**	**N lb/day**	**P lb/day**
Binghamton-Johnson	22.2	4200	2100 / 2100	1950 / 1950	3480	870
Endicott	9.2	1470	1470	5200 / 8800	2260	600
Chenango	2.2	400	400	1300 / 2800	710	165
Owego #2	2.8	360	360	1200 / 2500	640	150
Owego #1	0.7	200	200	830 / 1400	300	80
Owego Village	.97	100	100	415 / 700	150	40
Total	38.1	6730	4630 / 4630	10895 / 18150	7540	1905

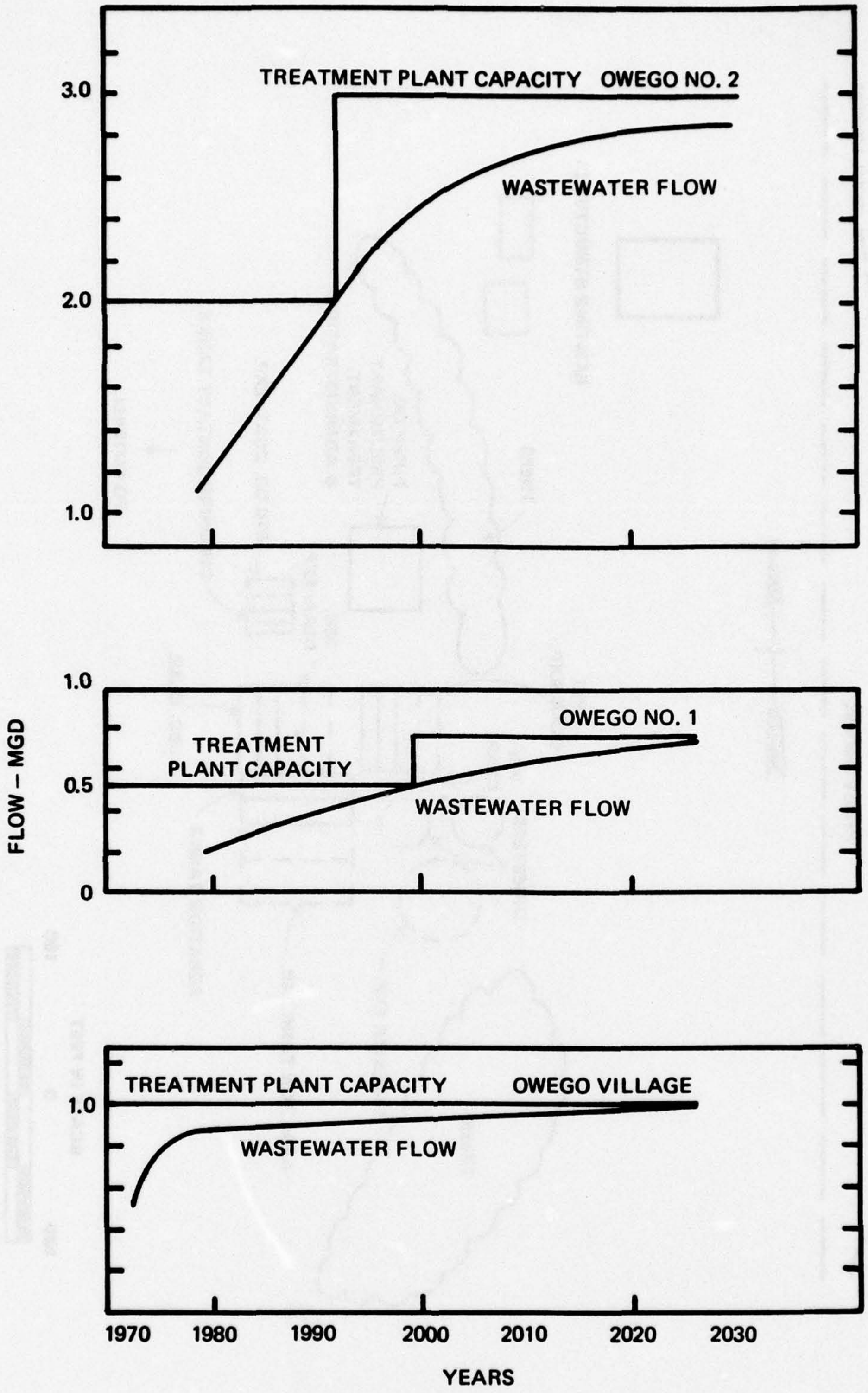
[1] Year 2020
[2] Warm Months/Cold Months



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – BROOME COUNTY

FIGURE VIII-36

PLAN 3A



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – TIOGA COUNTY

FIGURE VIII-37

PLAN 2A, 2B, 2C, 3A, 3B, and 3C

CHENANGO VALLEY STP
ALTERNATIVES 2B, 2C, 3B and 3C

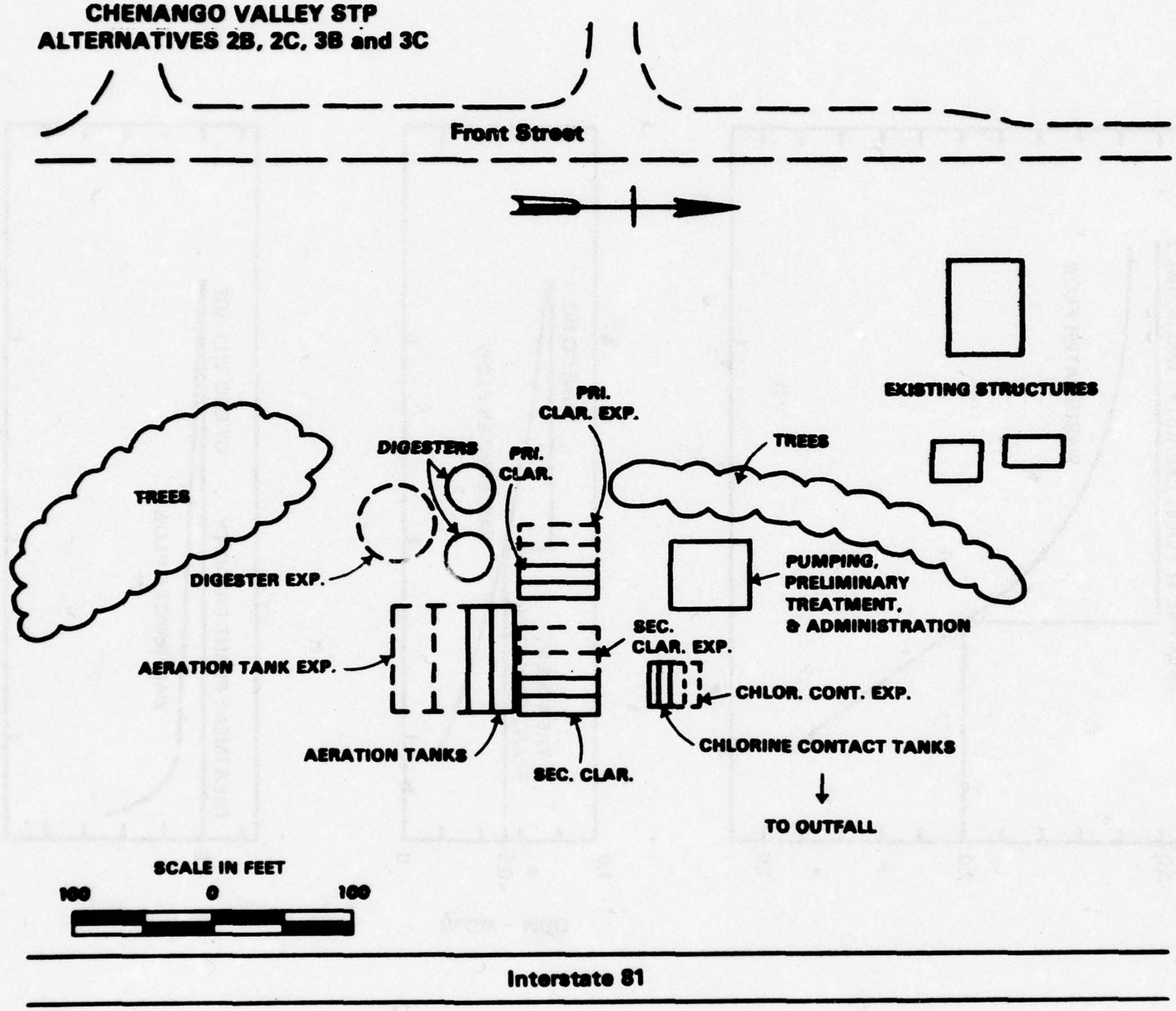


FIGURE VIII-38

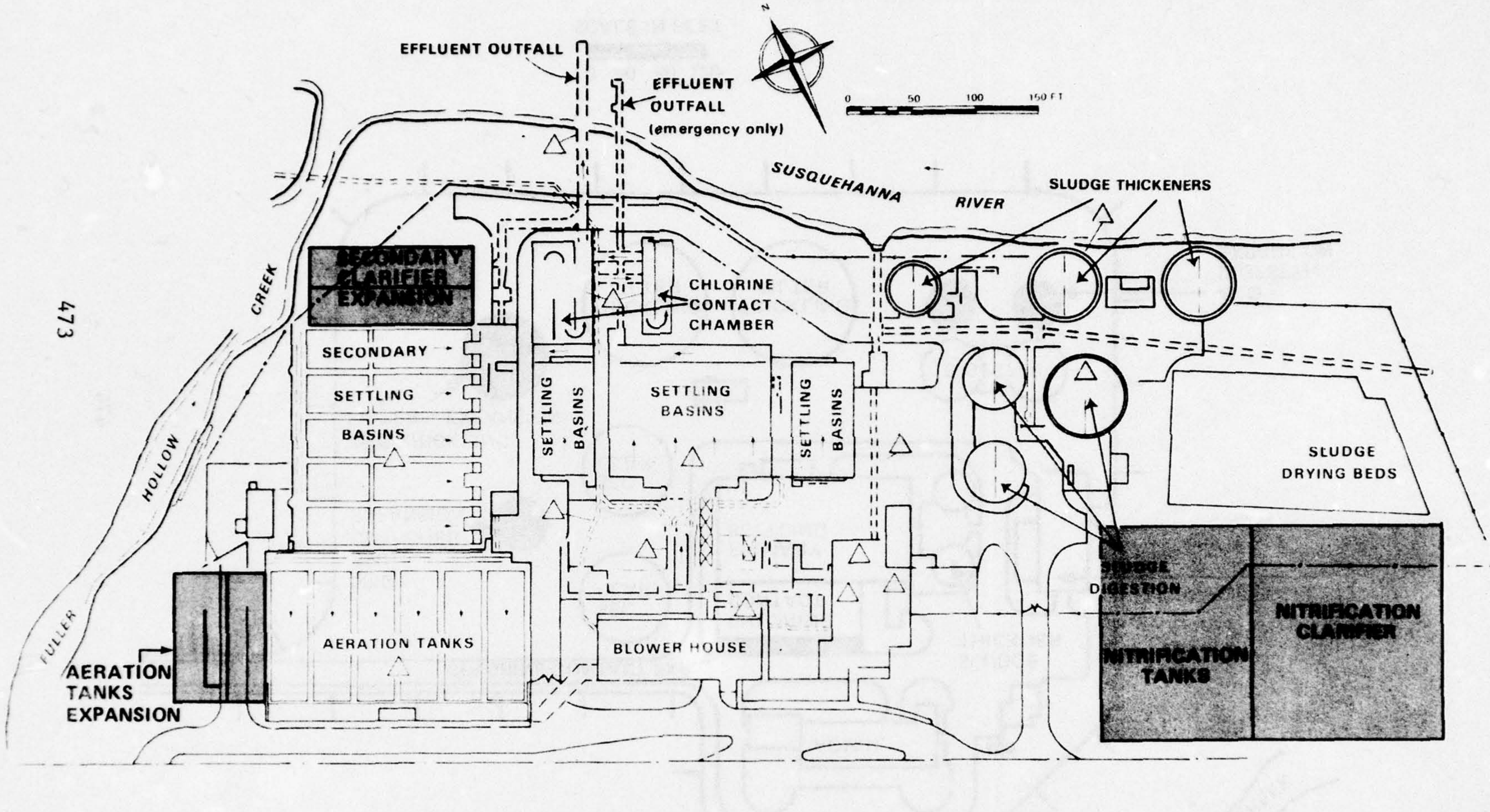


FIGURE VIII-39

BINGHAMTON-JOHNSON CITY STP

ALTERNATIVES 3A, 3B, 3C

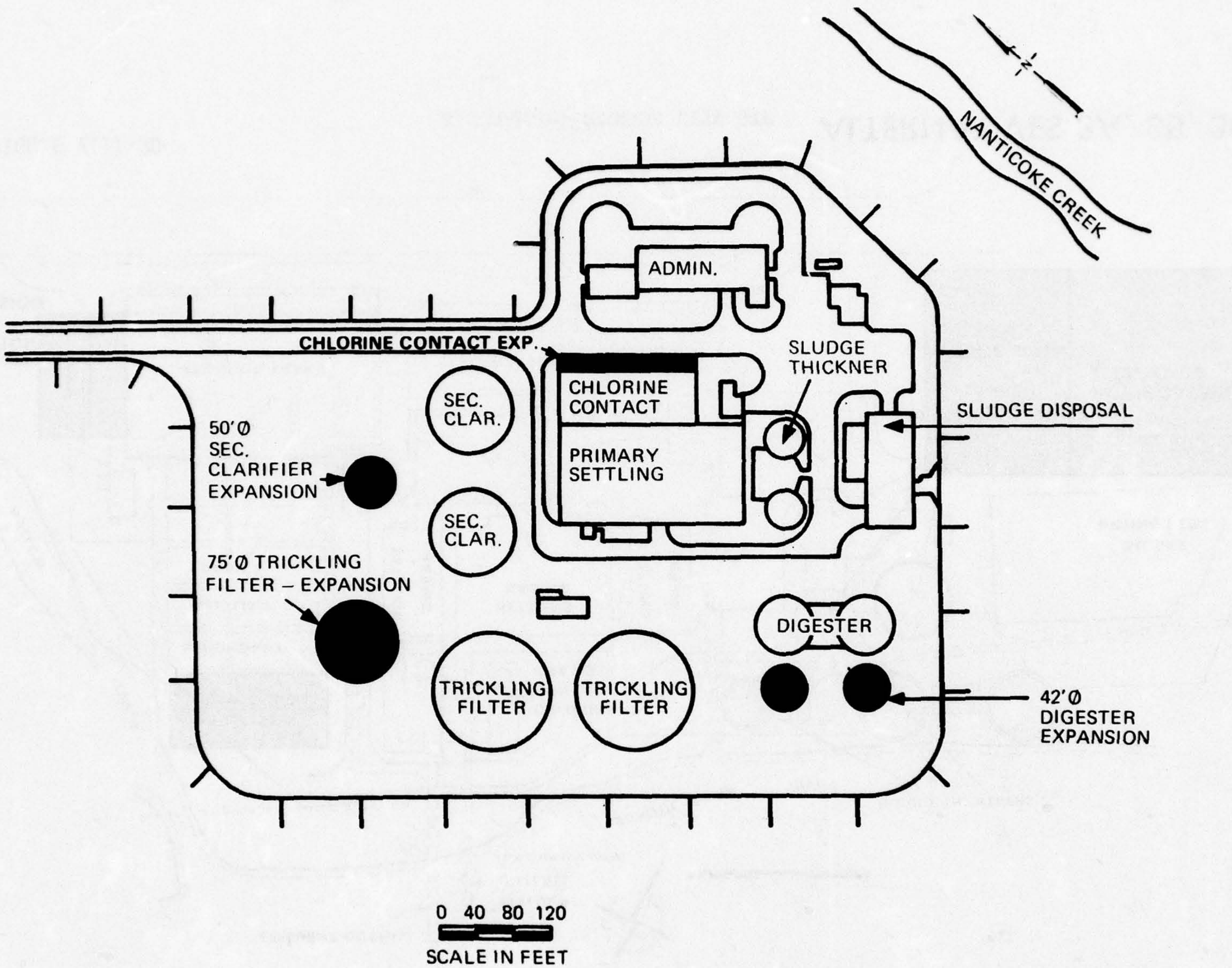
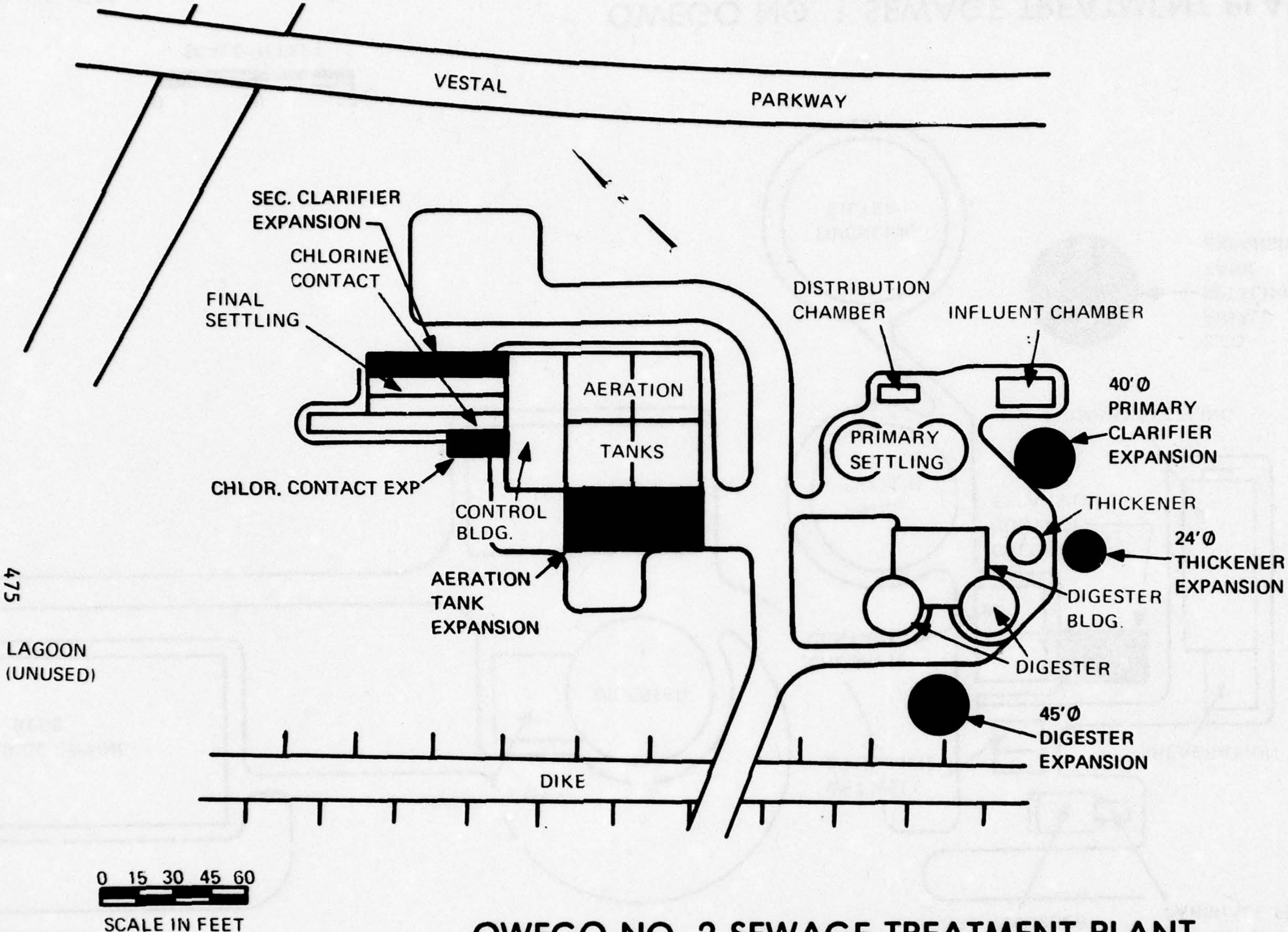


FIGURE VIII-49

ENDICOTT SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C



OWEGO NO. 2 SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-41

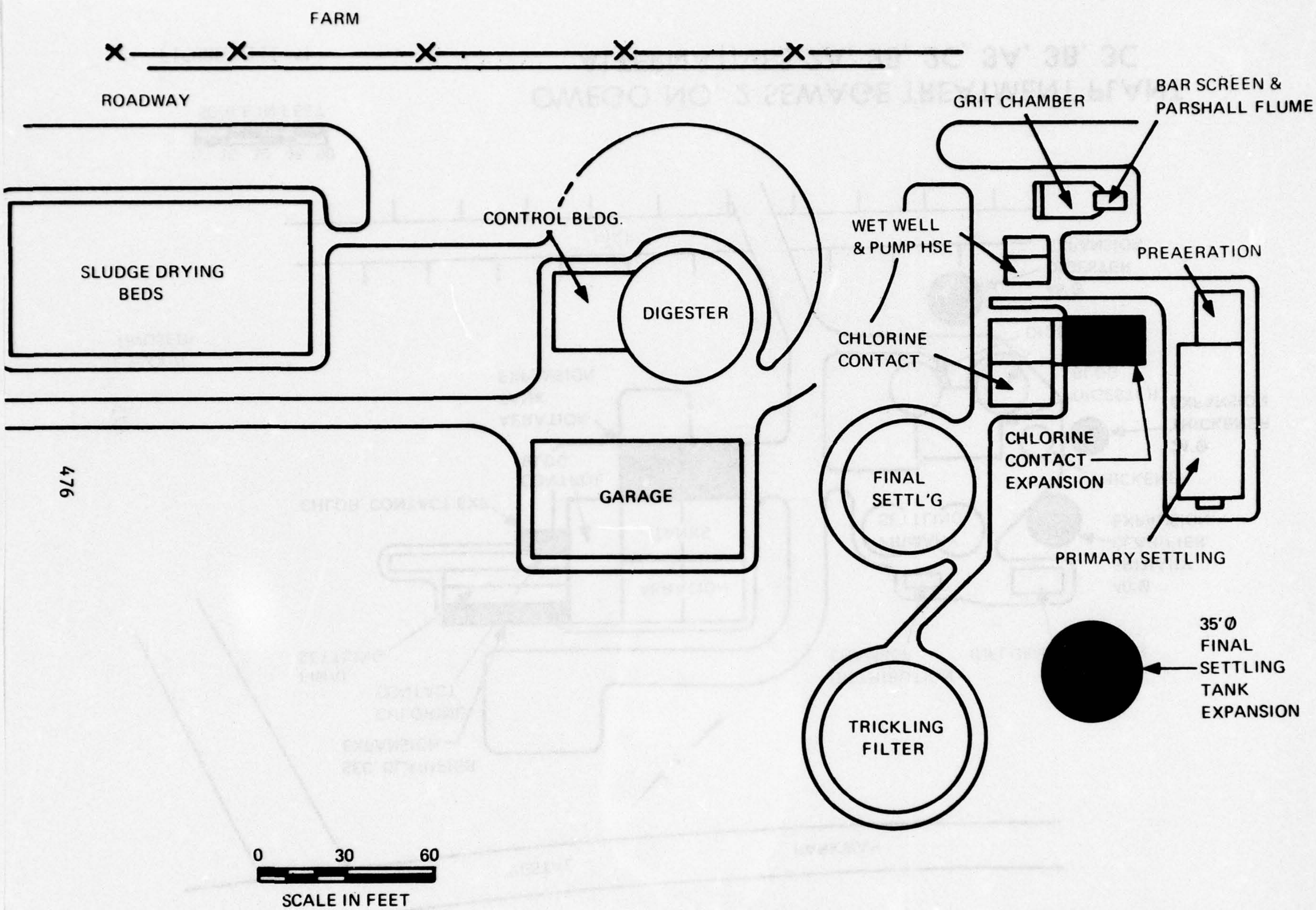


FIGURE VIII-42

OWEGO NO. 1 SEWAGE TREATMENT PLANT ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

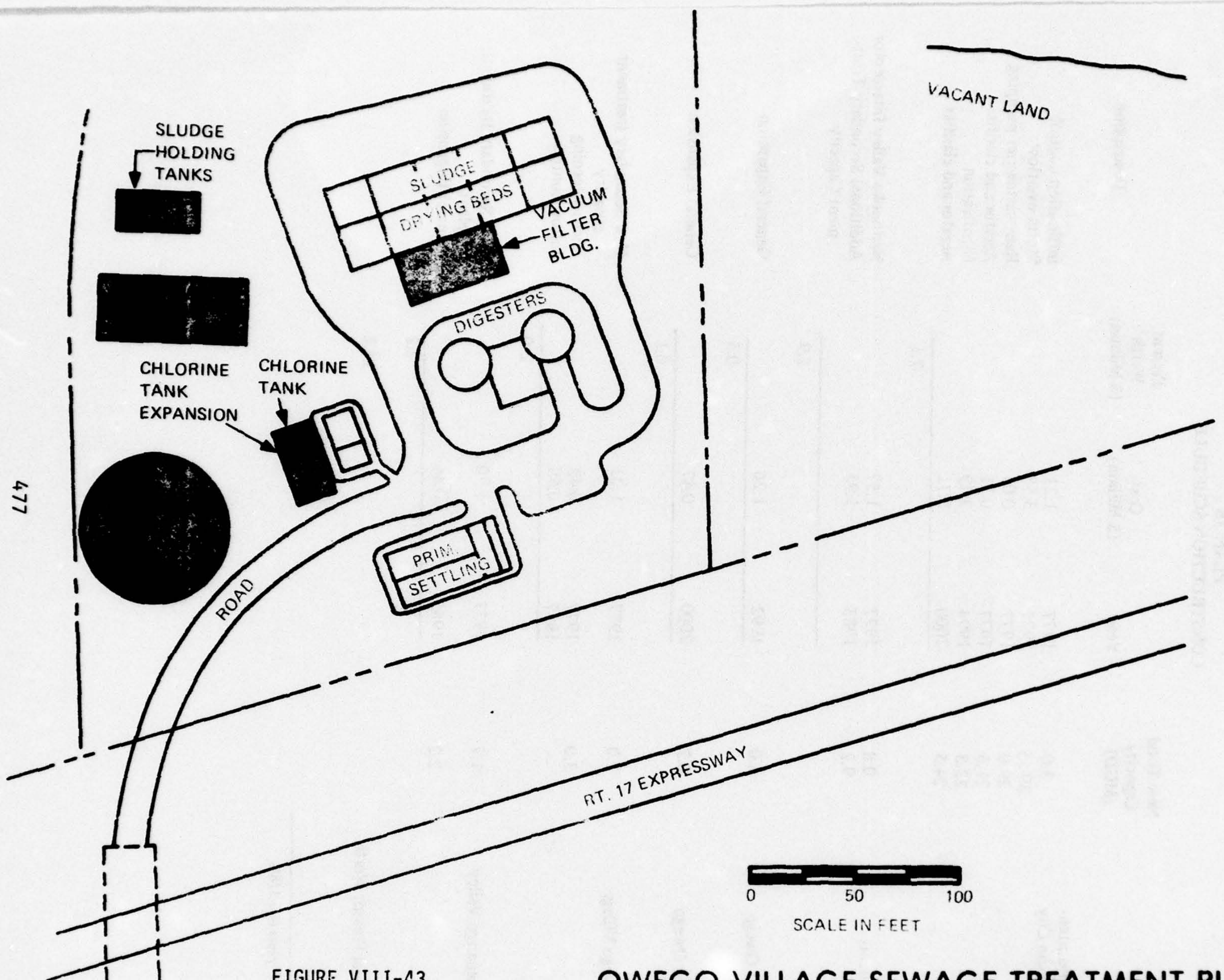


FIGURE VIII-43

OWEGO VILLAGE SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

TABLE VIII-54

PLAN 3B
CONSTRUCTION SCHEDULE

	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[1] ($ Million)	Description
Binghamton-Johnson City	3.0	1977	1.21		Infiltration control
	39.5	1977	3.58		Storm overflow
	29.0	1977	0.05		Raw wastewater pumping
	21.5	1977	0.71		Aerator and clarifier
	22.3	1994	3.79		Nitrification
	24.5	2000	0.71		Aerator and clarifier
				7.1	
Endicott	0.8	1977	1.57		Nanticoke Valley Interceptor
	9.2	1983	1.95		Additional Secondary Treatment Capacity
				2.9	
East Owego	3.0	1992	1.26		General Expansion
				0.5	
West Owego	0.7	2000	0.47		General Expansion
				0.1	
Owego Village	1.0	1977	1.02		New Secondary treatment capacity
	3.0	1977	0.42		Micro-screening
	–	1977	0.58		Inflow control
				2.0	
Chenango Valley	1.7	1977	1.90		New Secondary treatment plant
	2.2	1985	0.46		General Expansion
				2.2	
Total Present Worth				14.8	

[1] 50 years @ 6-1/8%

TABLE VIII-55

CAPITAL COST BREAKDOWN FOR AN AERATION TANK AND SECONDARY CLARIFIER SET AT THE BINGHAMTON–JOHNSON CITY STP*

1)	Clarifier	\$190,000
2)	Aeration Tank (30′ X 100′)	225,000
3)	Yard Piping & Renovation Instrumentation	80,000
4)	Additional Pumping and Valves	50,000
	Sub Total	545,000
	Eng. & Cont. @ 30%	164,000
	Total	\$709,000

*July 1975 ENR 2248

TABLE VIII-5C

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE SECONDARY TREATMENT
FACILITIES AT ENDICOTT STP FROM 7.7
TO 9.2 MGD CAPACITY*

1)	Preliminary Treatment	44,000
2)	Raw Wastes Pumps	74,000
3)	Trickling Filter (75′ϕ - 6′ deep)	105,000
4)	Secondary Clarifier (50′ϕ X 8′- 6 SWD)	158,000
5)	Chlorine Tank & Feed	51,000
6)	Digester Incl Bld Co. (42′ϕ - 30″ high)	440,000
7)	Sludge Pumping	84,000
8)	Piles (for all structures)	345,000
9)	Misc. Electricity, Heating and Ventilation	148,000
10)	Yard Piping	54,000
	Subtotal	1,503,000
	Engineering & Contingencies @ 30%	451,000
	Total	1,954,000

*July 1975 ENR 2248.

TABLE VIII-57

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 2 CAPACITY FROM 2.0 MGD
TO 3 MGD*

1)	Prelimination Treatment & R.W. Pumps, wet well	137,000
2)	Preliminary Settling Tanks 1 tank – 1250 SF	63,000
3)	Aeration Tank & Air System 31,000 CF	116,000
4)	Pumps – Recirculating Piping, Structure Upgrade	80,000
5)	Sludge Thickener (for 4 MGD 24′ Diameter)	42,000
6)	Digester 45′ ϕ X 20′ Diameter	220,000
7)	Chlorine Tank & Feed System	58,000
8)	Sludge Pumping	47,000
9)	Final Clarifiers 55′ X 12′ X 8.5′ depth	74,000
10)	Misc. – Electricity, Heating and Ventilation Sitework	132,000
	Subtotal	969,000
	30% Engr & Cont	291,000
	Total	1,260,000

*July 1975 ENR 2248.

TABLE VIII-53

**CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 1 CAPACITY FROM 0.5 MGD
TO 0.7 MGD***

1)	Preliminary Treatment	26,000
2)	R.W. Pumps, Internal Piping	90,000
3)	Final Settling Tank (35′ϕ, Dept = 8′)	74,000
4)	Chlorination Tank (16′ X 40′ X 6′)	47,000
5)	Misc. Electrical Instrumentation Heating and Vent.	58,000
6)	Yard Piping, Effluent Piping	68,000
	Subtotal	363,000
	30% Engr. & Cont.	109,000
	Total	472,000

*July 1975 ENR 2248.

TABLE VIII-59

CAPITAL COST BREAKDOWN
FOR UPGRADING THE VILLAGE OF OWEGO
STP SECONDARY TREATMENT CAPACITY
TO 1 MGD*

1)	Secondary Pump Station (Max Flow = 3.0)	200,000
2)	Trickling Filter A = 4,370 SF 75'ϕ X 5'-6" Stone Depth	87,000
3)	Final Settling TKS A = 1,904 SF 2 - 14' X 68' X 7' SWD	78,000
4)	Chlorine Tanks Vol = 3,800 16' X 40' X 6' SWD	25,000
5)	Sludge Holding Tanks 2 - 19' X 22' X 9' SWD With Mixers	60,000
6)	Vacuum Filter Area = 100 SF including Building 8'ϕ X 4' Face	160,000
7)	Preliminary Treatment – Screens, Grit removal Conninutor, etc.	65,000
8)	Site Work – Added Fill, Yard Piping, Grading, etc.	110,000
	Subtotal	785,000
	30% Engr. & Cont.	235,000
	Total	1,020,000

*July 1975 ENR 2248.

TABLE VIII-60

CAPITAL COST BREAKDOWN FOR SECONDARY TREATMENT PLANT AT CHENANGO VALLEY (Capacity of 1.7 mgd)*

1.	Preliminary Treatment	$ 119,000
2.	Primary Settling Tanks (A = 2145 SF 3 - 13' x 55' x 8' SWD)	108,000
3.	Aeration Tanks (V = 57,600 CF 3 - 20' x 80' x 12' SWD)	146,000
4.	Diffused Aeration System	246,000
5.	Secondary Settling Tanks (A = 2436 SF 3 - 14' x 58' x 10' SWD)	116,000
6.	Sludge Digestion (V = 68,000 CF 2 - 46'ϕ)	360,000
7.	Chlorine Contact Tanks (V = 5880 CF 3 - 8' x 35' x 7' SWD)	37,000
8.	Chlorine Feed Equipment	40,000
9.	Yardwork	175,000
10.	Outfall to Chenango River	115,000
		$1,462,000
	30% Engr. & Cont.	438,000
	TOTAL	$1,900,000

*July 1975 ENR 2248

TABLE VIII-60 (Continued)

(Expansion of STP Capacity from 1.7 mgd to 2.2 mgd)

1.	Preliminary Treatment	$ 29,000
2.	Primary Settling Tank (A = 605 SF 1 = 11' x 55' x 8' SWD)	33,000
3.	Aeration Tank (V = 16,300 CF 1 - 17' x 80' x 12' SWD)	30,000
4.	Diffused Aeration Tank	55,000
5.	Secondary Settling Tank (A = 696 SF 1 - 12' x 58' x 10' SWD)	35,000
6.	Sludge Digestion (V = 16,000 CF 1 - 32' ϕ)	129,000
7.	Chlorine Contact Tank (V = 1715 CF 1 - 7' x 35' x 7' SWD)	9,000
8.	Chlorine Feed Equipment	13,000
9.	Yardwork	21,000
		$354,000
	+ 30% Engr. & Cont.	106,000
	TOTAL	$460,000

TABLE VIII-51

CAPITAL COST BREAKDOWN
FOR THE NANTICOKE INTERCEPTOR
(AVERAGE DESIGN FLOW = 0.8 M.G.D.*
PEAK DESIGN FLOW = 2.3 M.G.D.)

1) 18" sewer – 25,000' @ $38/LF	957,000
2) 110 manholes @ $1,100	120,000
3) Misc. items	33,000
4) Railroad crossing 27" casing 100' @ $270	27,000
5) Creek Crossing 200/LF @ $350/LF	70,000
Subtotal	1,207,000
30% Engr. & Contr.	362,000
Total	1,569,000

*use 18" ϕ sewer @ S = 0.0012 min. assume avg. depth = 12-14 feet.

July 1975 ENR 2248.

TABLE VIII-62

PLAN 3B
OPERATION AND MAINTENANCE COSTS

	First Year	Last Year	Description	Annual Cost ($ Million/Yr)	Present Worth[1] ($ Million)
Binghamton-Johnson City	1977	2026	Secondary	0.72	
	1977	2026	Storm Overflow	0.05	
	1994	2026	Nitrification	0.17	
					12.8
Endicott	1977	1982	Secondary	0.31	
	1983	2026	Secondary	0.36	
					5.1
Chenango	1977	1984	Secondary	0.13	
	1985	2026	Secondary	0.15	
					2.1
East Owego	1977	1991	Secondary	0.14	
	1992	2026	Secondary	0.19	
					2.4
West Owego	1977	1999	Secondary	0.04	
	2000	2026	Secondary	0.05	
					0.6
Owego Village	1977	2026	Secondary	0.07	
	1977	2026	Microscreening	0.002	
					1.1
				Total Present Worth:	24.1

[1] 50 years @ 6-1/8%.

TABLE VIII-63

PLAN 3B

CAPITAL COSTS (MILLION $) AND OPERATION AND MAINTENANCE COSTS ($$10^3$/Yr)

Year	Binghamton Johnson Capital	Binghamton Johnson O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	E. Owego Capital	E. Owego O&M	W. Owego Capital	W. Owego O&M	Owego Village Capital	Owego Village O&M
1977	5.55	770	1.90	130	1.57	310		140		40	2.0	70
1983					1.95	360						
1985			.46	150								
1989												
1990												
1992							1.26	190				
1994	3.79	940										
2000	.71								.47	50		
2010												
2018												
2026		940		150		360		190		50		70
Present Worth Million $	7.1	12.8	2.2	2.1	2.9	5.1	.5	2.4	.1	.6	2.0	1.1
Present Worth of Replacement Million $	6.4		.5		3.8		.8		.2		.5	
Total Present Worth Million $	26.3		4.8		11.8		3.7		0.9		3.6	

TABLE VIII-64

PLAN 3B COSTS – PRESENT WORTH[1]

	Chenango Valley	Binghamton-Johnson City	Endicott	East Owego	West Owego	Owego Village	Total
Wastewater Treatment							
Capital	1.7	2.3	1.3	0.5	0.1	1.0	6.9
O & M	1.5	8.2	4.4	1.9	0.4	0.7	17.1
Replacement	0.4	5.2	3.6	0.7	0.1	0.3	10.3
TOTAL	3.6	15.7	9.3	3.1	0.6	2.0	34.3
Interceptors							
Capital	–	–	1.6	–	–	–	1.6
O & M	–	–	–	–	–	–	–
Replacement	–	–	–	–	–	–	–
TOTAL			1.6				1.6
Storm Overflows							
Capital	–	3.6	–	–	–	1.0	4.6
O & M	–	0.8	–	–	–	0.1	0.9
Replacement	–	0.9	–	–	–	0.1	1.0
TOTAL	–	5.3	–	–	–	1.2	6.5
Infiltration Control							
Capital	–	1.2	–	–	–	–	1.2
O & M	–	–	–	–	–	–	–
Replacement	–	–	–	–	–	–	–
TOTAL		1.2					1.2
Sludge							
Capital	0.465	0.05	0.01	0.005	0.001	0.002	0.5
O & M	0.6	3.8	0.7	0.5	0.2	0.3	6.1
Replacement	0.1	0.3	0.2	0.1	0.1	0.1	0.9
TOTAL	1.2	4.1	0.9	0.6	0.3	0.4	7.5
Totals							
Capital	2.2	7.1	2.9	0.5	0.1	2.0	14.8
O & M	2.1	12.8	5.1	2.4	0.6	1.1	24.1
Replacement	0.5	6.4	3.8	0.8	0.2	0.5	12.2
TOTAL PLAN COST	4.8	26.3	11.8	3.7	0.9	3.6	51.1

[1] Based on a 50 year economic life @ 6-1/8% and ENR = 2248.

PLAN 3C

The intent of this plan is to maintain minimum dialy average DO levels above 5 mg/l in the Susquehanna River during low flow conditions and above a minimum instantaneous DO level of 4 mg/l during storm overflow conditions. Plans 3A, 3B, and 3C are similar except for the degree of regionalization.

The only additional requirement to maintain a minimum daily average of 5 mg/l DO in the Susquehanna River (Plan 3) over the 4 mg/l system (Plan 2) is the provision of nitrification facilities for ammonia removal at the Binghamton-Johnson City STP during the 1990's. Other than the modification in capital improvements and manpower requirements (see Institutional Analysis Appendix), the detailed refinement of Plan 3C is virtually identical to Plan 2C.

MUNICIPAL WASTEWATER MANAGEMENT

Regionalization of STP's

The Broome County plants would be the existing Binghamton-Johnson City and Endicott STP's and a new STP for a limited Chenango Valley service area. The first phase of this Plan would serve a limited Chenango Valley wastewater management area. In the final phase, the treatment plant and service area would be expanded to include the entire wastewater management area.

Plan 3C is identical to Plan 3B except that the scope of sewerage service for Chenango Valley would be limited to a smaller area for five years. Initial treatment plant capacity in 1977 would be 1.0 mgd (vs. 1.7 mgd for Plan 2B). After 1982, the entire Chenango Valley service area would be sewered and the STP expanded to 2.2 mgd. Tioga County plants would be the Town of Owego STP's No. 1 (West Owego) and No. 2 (East Owego) and the Village of Owego STP (all existing).

Service area and interceptors in this plan are shown in Plate 5. With the exception of the Chenango Valley sewerage system, these are the same interceptors included in the Baseline Plan. Plate 6 details the phasing of the Chenango Valley service area.

Treatment Levels and Processes

Nitrification facilities would be added to the existing activated sludge secondary treatment process at the Binghamton-Johnson City STP by 1994. Secondary treatment level would be sufficient at the other five STP's. The new Chenango Valley STP would use an activated sludge process. The Owego Village treatment plant was assumed to be upgraded to provide secondary treatment using a trickling filter process (recently changed to proposal for an activated sludge process). The activated sludge process at the Town of Owego STP No. 2 and the trickling filter processes at the Village of Endicott and the Town of Owego STP No. 1 would continue in use with their capacities upgraded as the wastewater flow increases.

Infiltration Control Level in the City of Binghamton

For the level of treatment (secondary plus nitrification) and the regionalization scheme assumed in this plan, an infiltration reduction of 3 mgd was found economically justifiable.

Nonstructural Measures for Flow Reduction

In this plan, the Village of Endicott and the Town of Owego would institute metered user rates. The increase in price the consumer would be paying should result in a reduction of water consumption, and, therefore, wastewater flows.

An educational program to encourage the use of water saving devices could result in an estimated 20 percent reduction in the projected incremental increase in per capita flows. Although such a program could be undertaken, no formal educational activities have been specified in this plan.

STORM OVERFLOW MANAGEMENT

Microscreening units followed by chlorination would be provided near five overflow sites in Binghamton (see Figure VIII-44). This system would maintain a minimum instantaneous DO above 4 mg/1 during most storm conditions. A detailed description is given in Chapter VII. An estimate was also made of the costs of inflow control and microscreening treatment for the Village of Owego combined sewer system.

SLUDGE MANAGEMENT

Several sludge management techniques have been analyzed in Chapter VI. The land application of liquid sludge is recommended, with landfill of dewatered sludge provided as a backup.

PERFORMANCE

Impact on Susquehanna River dissolved oxygen levels:

1. The minimum daily average dissolved oxygen level will be between 5 and 6 mg/l during low river flow and dry wastewater flow conditions.

2. The minimum instantaneous dissolved oxygen level will be between 4 and 5 mg/l during design storm conditions.

Pollutant mass loads discharged to the Susquehanna and Chenango Rivers by the STP's proposed in this alternative are summarized in Table VIII-65.

CONSTRUCTION SCHEDULE

Expansion of the existing wastewater facilities would be required during the planning period to handle the increase in projected flows (see Figures VIII-45 and VIII-46).

The Chenango Valley phased treatment plant would be built by 1977 to conform with the Federal requirements for secondary treatment by utilizing an activated sludge process (see Figure VIII-47). The plant would have an initial capacity of 1.0 mgd to be expanded to 2.2 mgd by 1982, with sewering extended to the entire Chenango Valley service area.

At the Binghamton-Johnson City STP, the raw wastewaters pumping capacity will be increased from 18.3 mgd to 29 mgd by the year 1977. Also by this year, one aerator and clarifier unit will be added to increase the capacity from 18.3 mgd to 21.5 mgd, with another aeration and clarifier unit to be added by the year 2000 to increase the capacity from 21.5 mgd to 24.5 mgd.

In addition to the secondary treatment expansions, nitrification will be required in 1994 to maintain minimum daily average DO above 5.0 mg/l at critical conditions. Suspended

growths nitrification (see Chapter V) will be used with capacities to last till the end of the planning period. The expansions for Plans 3B and 3C are virtually identical to those for Plan 3A with the exception being that the nitrification units are slightly smaller. All other treatment plants, i.e., Endicott, Owego Town No. 1 and 2, and Owego Village will have expansion schedule identical to Plan 3A. Additions to the Binghamton-Johnson City STP are shown in Figure VIII-48.

Storm overflow control facilities are assumed to be constructed at the City of Binghamton by 1977. Five microscreening facilities having a total treatment capacity of 39.5 mgd will be located as shown in Figure VIII-44.

At Endicott STP, the secondary treatment units will need expansion from 7.7 mgd to 9.2 mgd capacity by 1983. The trickling filters, secondary clarifiers, digesters, chlorination tank, raw wastewater and sludge pumping capacities will be expanded (see Figure VIII-49).

The Nanticoke service area wastewater will be transmitted to the Endicott STP through an 18-inch diameter sewer that has a design average flow of 0.8 mgd and a peak flow of 2.3 mgd. The sewer length is 25,000 feet. This interceptor will be built by the year 1977.

At the Town of Owego STP No. 2, the plant treatment capacity will be upgraded from the present 2.0 mgd to 3.0 mgd by 1992. The additional units include preliminary treatment, a primary settling tank with a 1,250 SF capacity; a 31,000 CF aeration tank and air system; a sludge thickener for 4 mgd; a digester having 45' diameter by 20' depth; a chlorination tank 25' by 18' and 8', and two final clarifiers having a 3 mgd capacity, each being 55' by 12' and 8.5' deep (see Figure VIII-50).

At the Town of Owego STP No. 1, the plant capacity will be expanded by the year 2000 from the present 0.5 mgd to 0.7 mgd. The added facilities will include a preliminary treatment capacity, a final settling tank, and a chlorination tank (see Figure VIII-51).

At the Village of Owego, the treatment facilities will be upgraded to provide secondary treatment by 1977. The capacity of the upgraded facility will be 1.0 mgd. The facilities added will include a trickling filter with a surface area of 4,370 SF, 75' diameter and 5'6" in depth; a secondary pump station designed to handle a maximum flow of 3.0 mgd; two final settling tanks each 14' by 68' and 7' deep; a

chlorine tank, volume 3,800 CF; two sludge holding tanks with mixers, each 19' by 22' and 9' deep; and a vacuum filter with an area of 100 SF (see Figure VIII-52).

At the Village of Owego, a microscreening treatment unit, with a capacity of 3 mgd, will be built by 1977 to provide treatment of storm overflows. Also, steps will be taken to control direct stream inflow into the sewer system.

A summary of the c nstruction schedule of the wastewater treatment facilities needed at each service area is given in Table VIII-66.

COST ANALYSIS

The capital costs breakdown for the expansion and/or upgrading of the existing treatment facilities at Binghamton-Johnson City, the Village of Endicott, the Town of Owego STP No. 1, the Town of Owego STP No. 2, and the Village of Owego are shown in Tables VIII-67 through VIII-71. Also, Tables VIII-72 and VIII-73, break down the capital costs for the new Chenango Valley STP and the Nanticoke interceptor, respectively.

The detailed operation and maintenance costs of the wastewater treatment plants for each service area during the planning period are given in Table VIII-74.

The capital, O&M, and replacement expenditures for each service area are summarized in Table VIII-75. Also shown in the table are the years these expenditures will be incurred. These correspond to the construction schedule described in the previous section. The replacement costs were estimated using an equivalent annual sinking fund approach.

The present worth of all the expenditures during the 50-year planning period is given in Table VIII-76 for each service area. The present worth costs for this plan are: capital -- $14.8 million, O&M -- $24.1 million, and replacement -- $12.2 million, for a total cost of $51.1 million.

CITY OF BINGHAMTON
LOCATION OF PLANNED OVERFLOW CONTROL FACILITIES

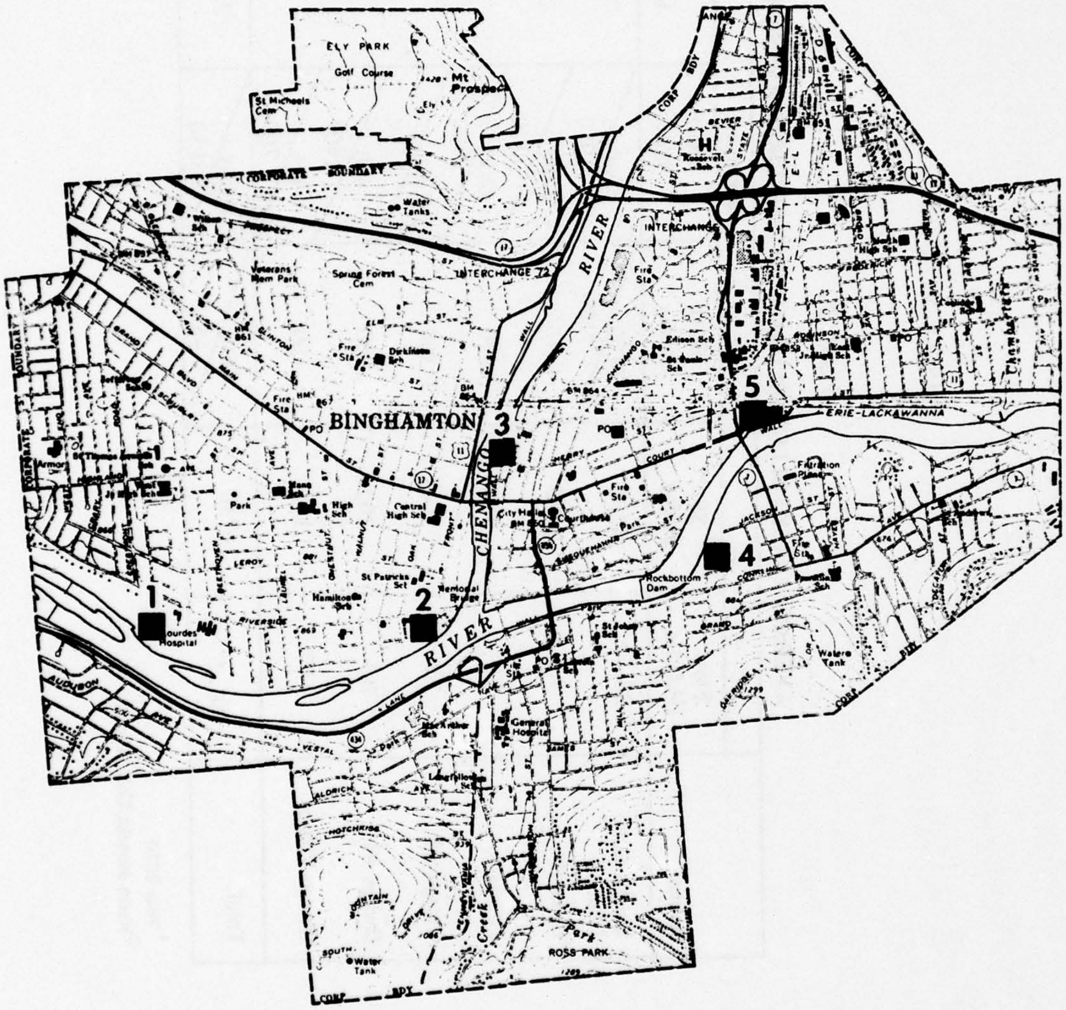


FIGURE VIII-44

TABLE VIII-65

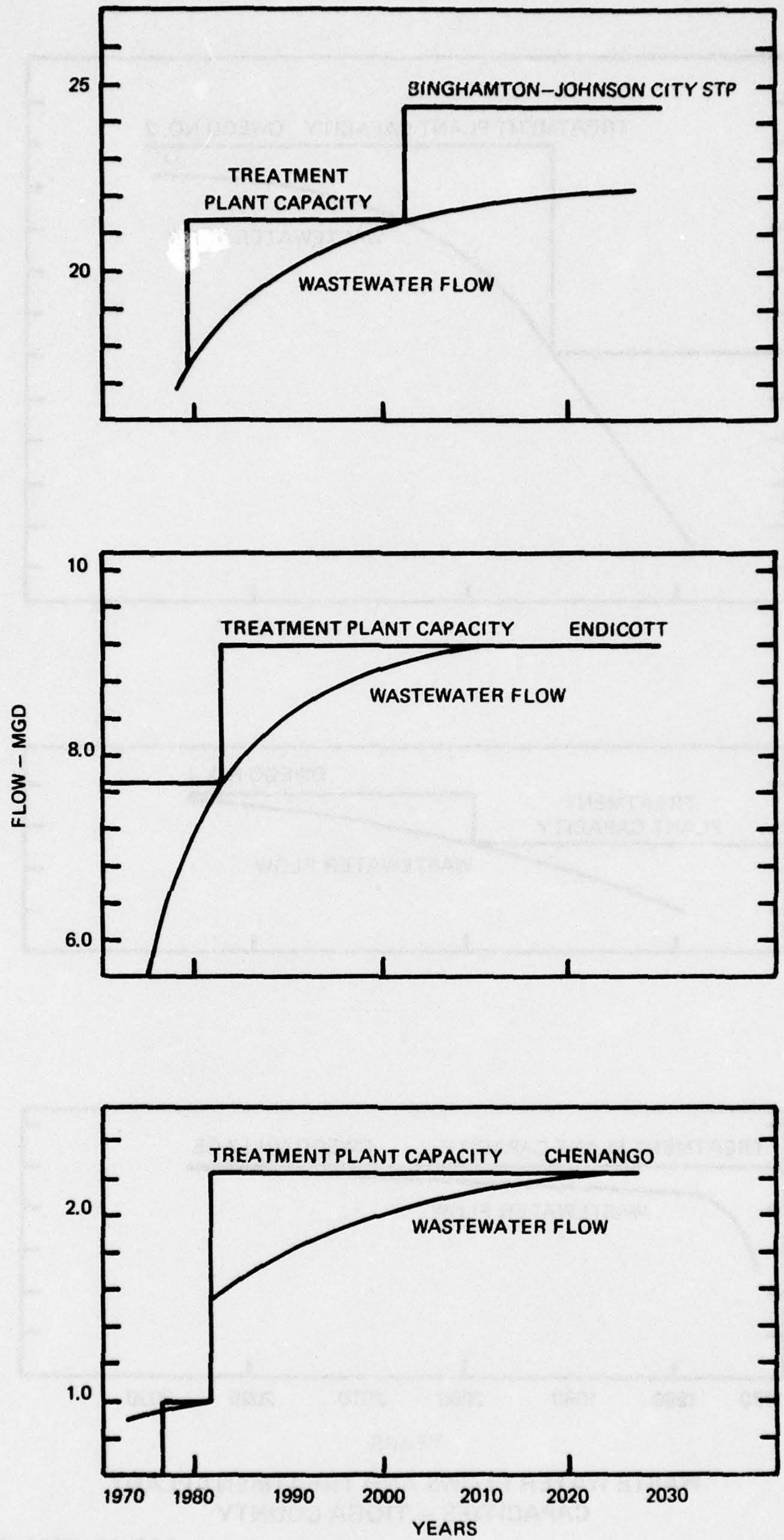
PLAN 3C

AVERAGE DAILY POLLUTION LOADS TO THE SUSQUEHANNA RIVER[1]

Source	Parameter					
	Flow (M.G.D)	SS lb/day	BOD[2] lb/day	NOD[2] lb/day	N lb/day	P lb/day
Binghamton-Johnson	22.2	4200	2100 / 2100	1950 / 1950	3480	870
Endicott	9.2	1470	1470	5200 / 8800	2260	600
Chenango	2.2	400	400	1300 / 2800	710	165
Owego #2	2.8	360	360	1200 / 2500	640	150
Owego #1	0.7	200	200	830 / 1400	300	80
Owego Village	.97	100	100	415 / 700	150	40
Total	38.1	6730	4630 / 4630	10895 / 18150	7540	1905

[1] Year 2020

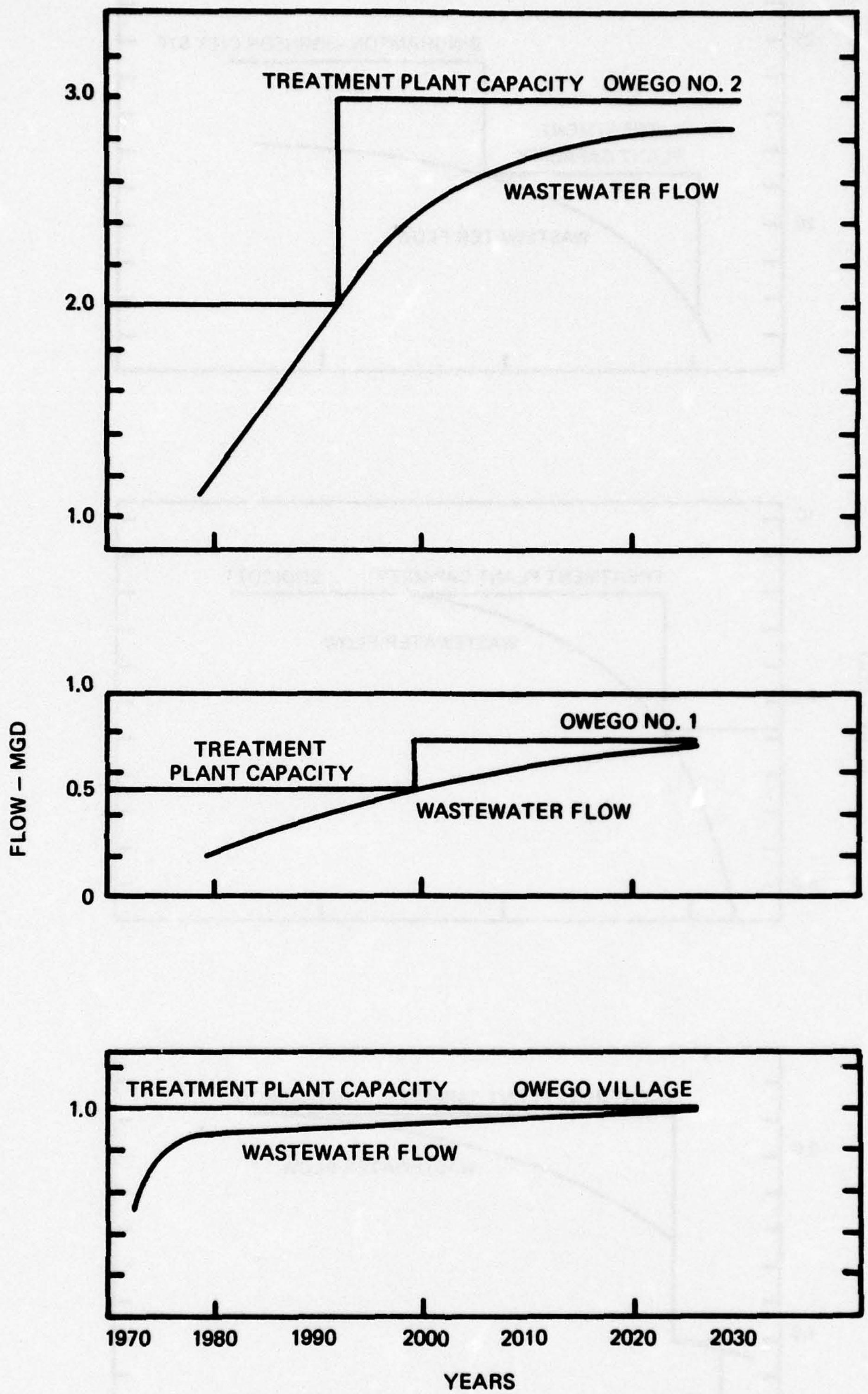
[2] Warm Months/Cold Months



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – BROOME COUNTY

PLAN 3C

FIGURE VIII-45



WASTE WATER FLOWS AND TREATMENT PLANT CAPACITIES – TIOGA COUNTY

FIGURE VIII-46

PLAN 2A, 2B, 2C, 3A, 3B, and 3C

CHENANGO VALLEY STP
ALTERNATIVES 2B, 2C, 3B and 3C

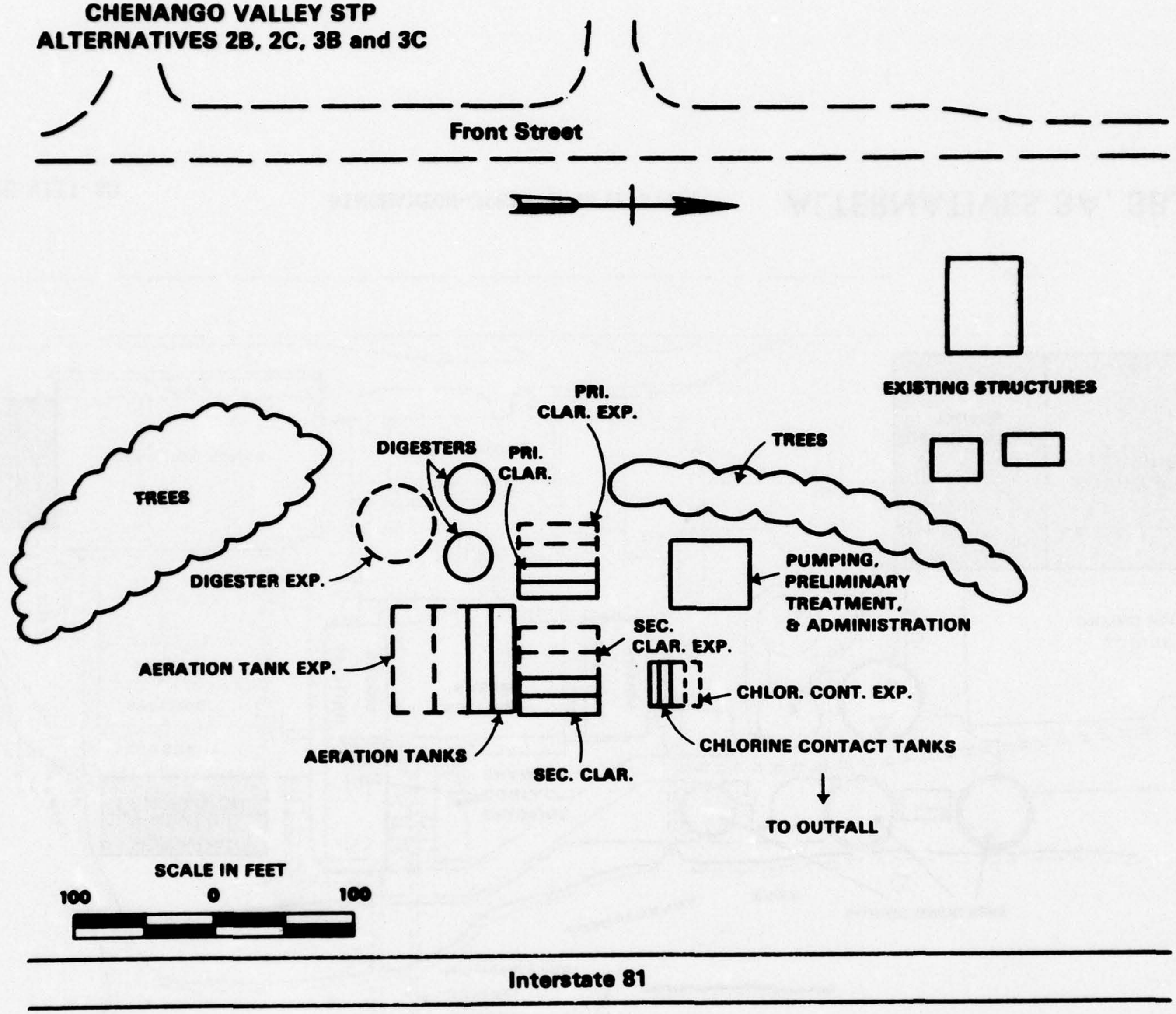


FIGURE VIII-47

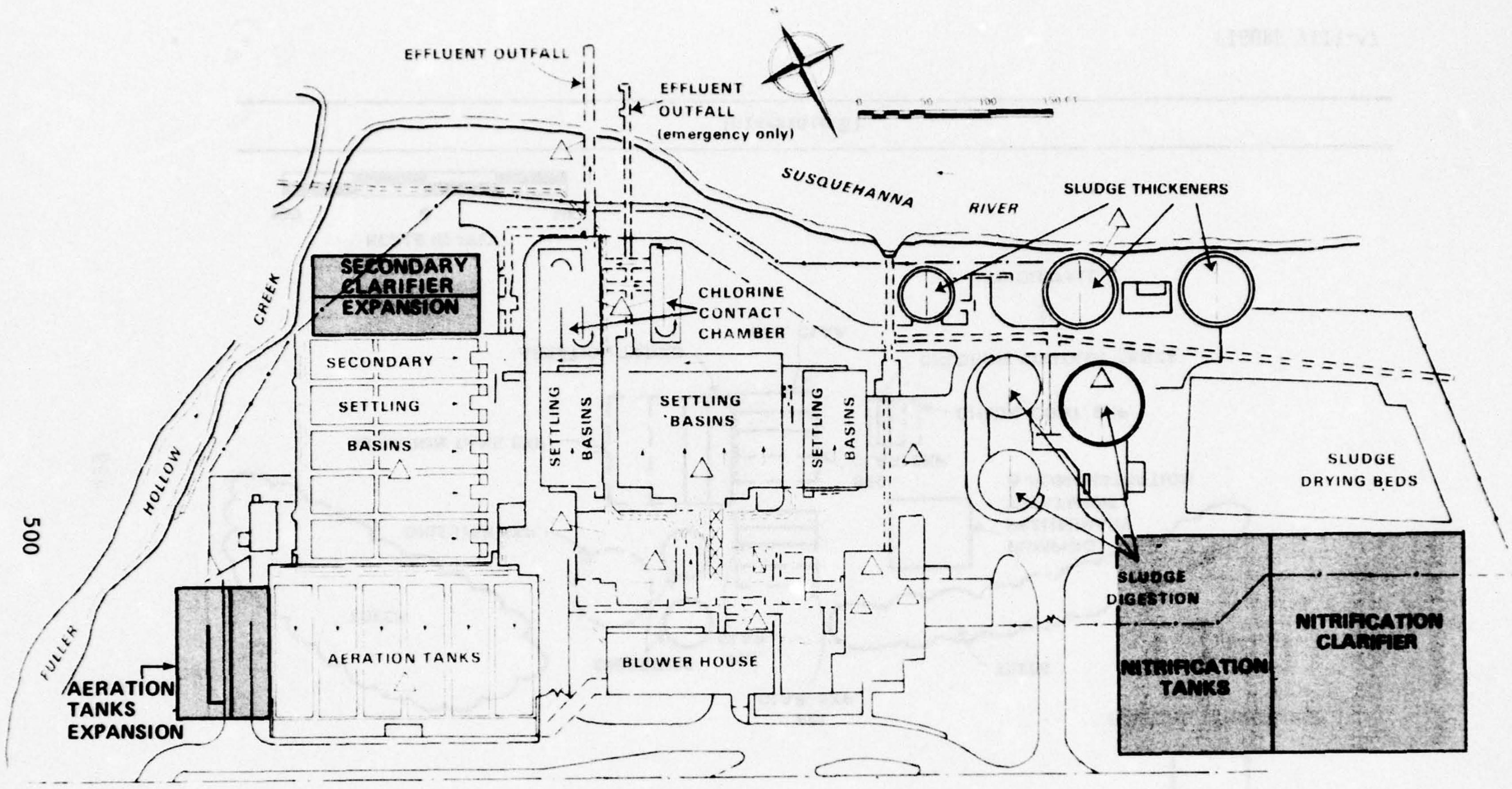


FIGURE VIII-48

BINGHAMTON-JOHNSON CITY STP

ALTERNATIVES 3A, 3B, 3C

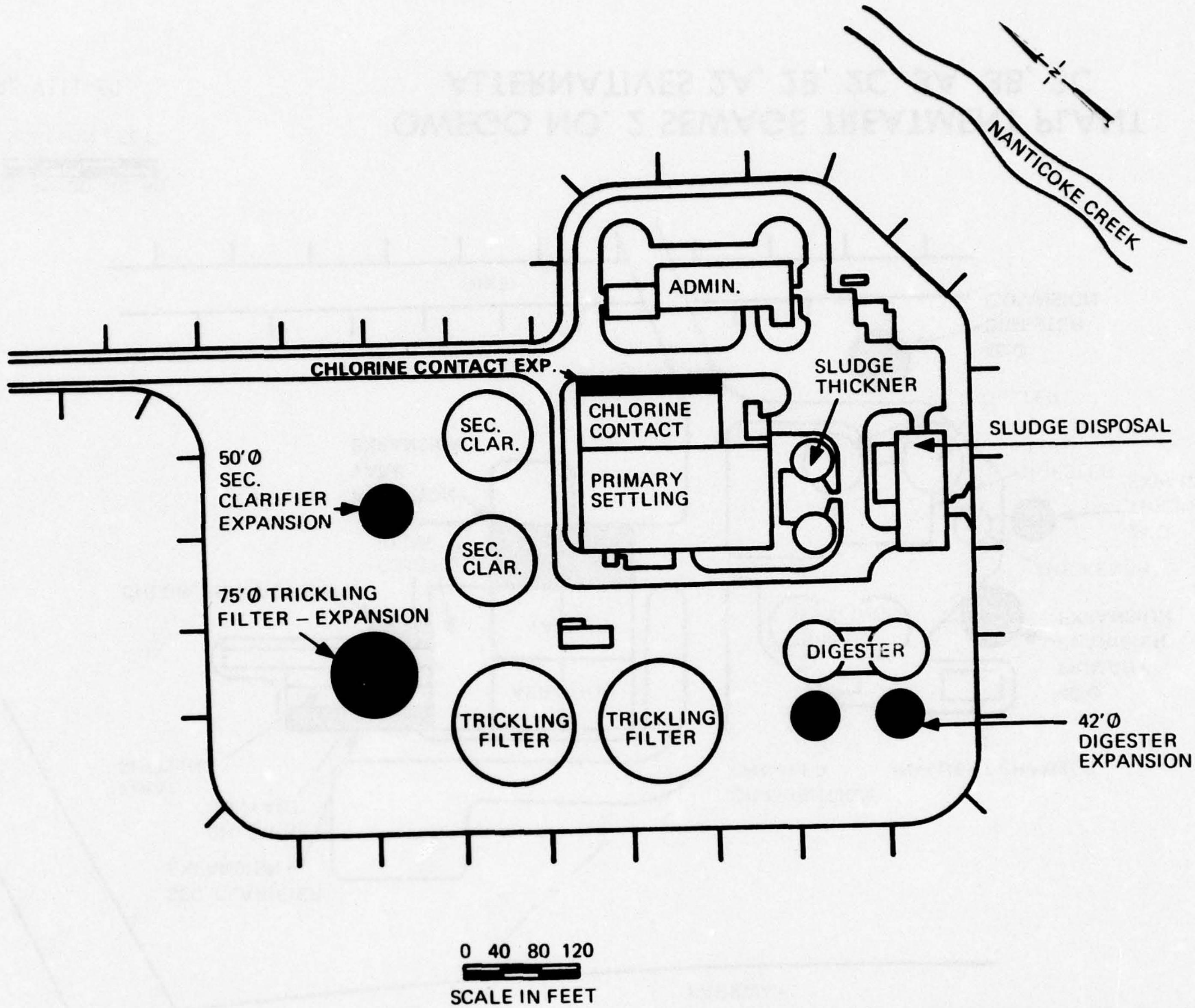
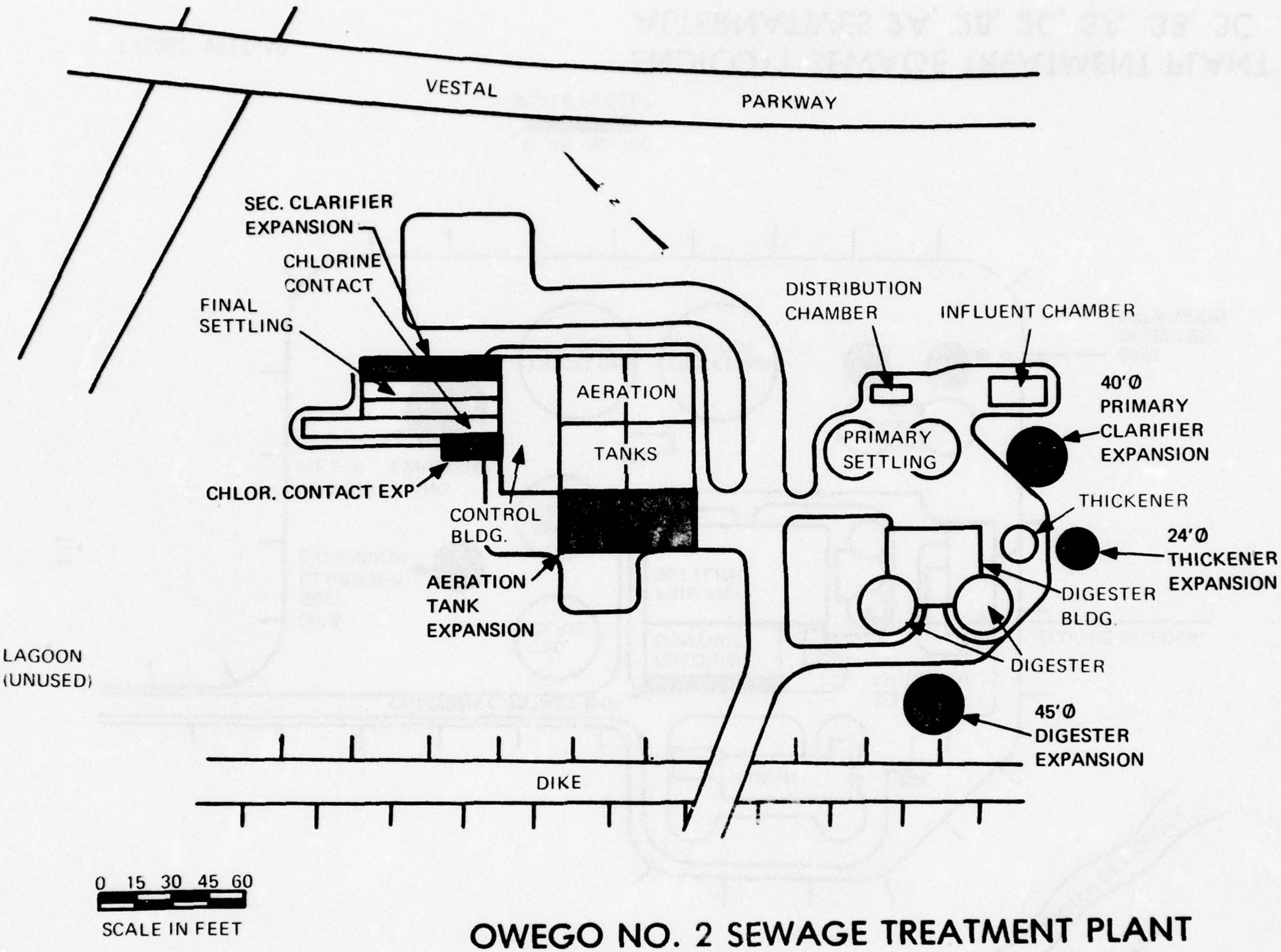


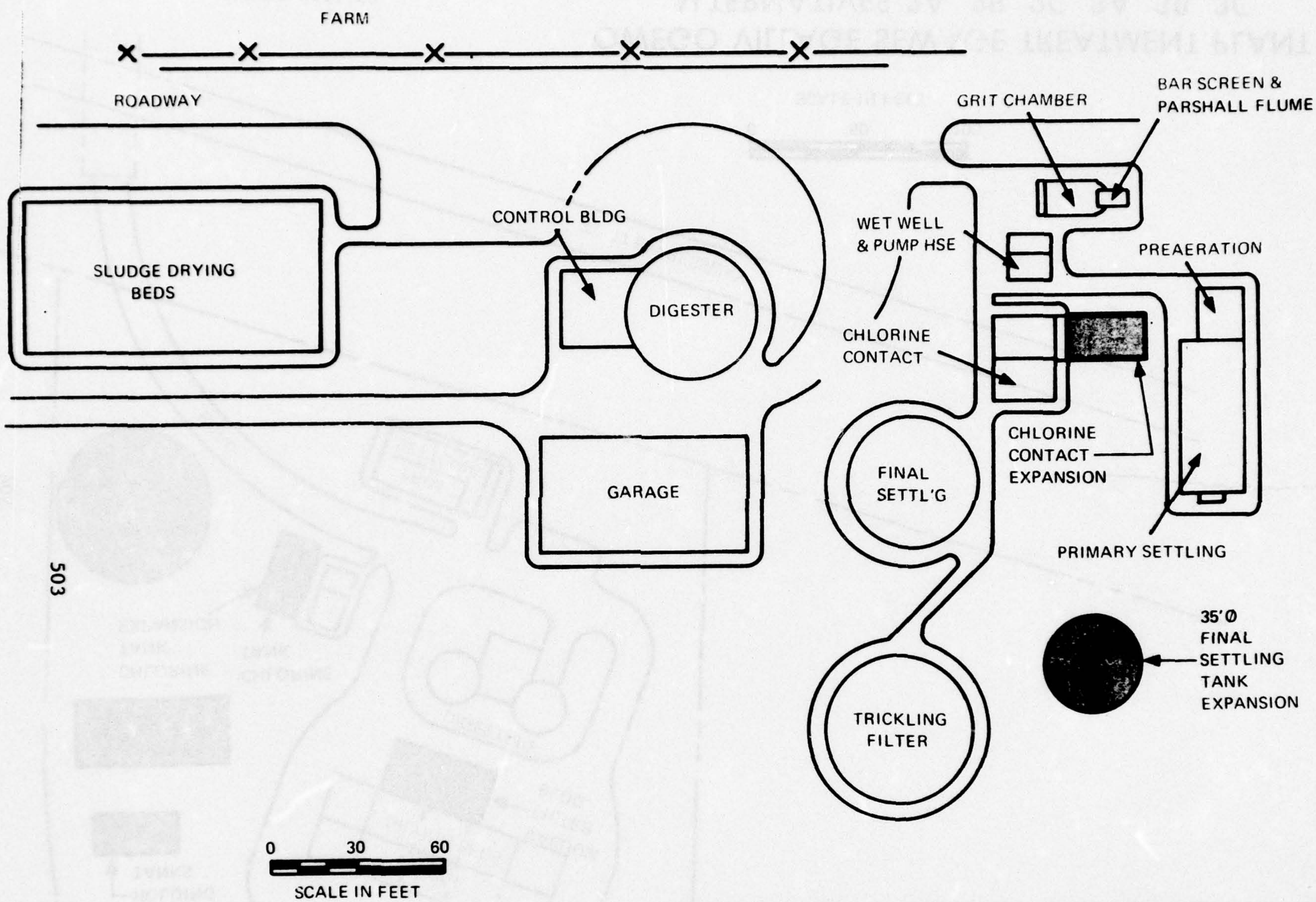
FIGURE VIII-49

ENDICOTT SEWAGE TREATMENT PLANT ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C



OWEGO NO. 2 SEWAGE TREATMENT PLANT ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-59



OWEGO NO. 1 SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

FIGURE VIII-51

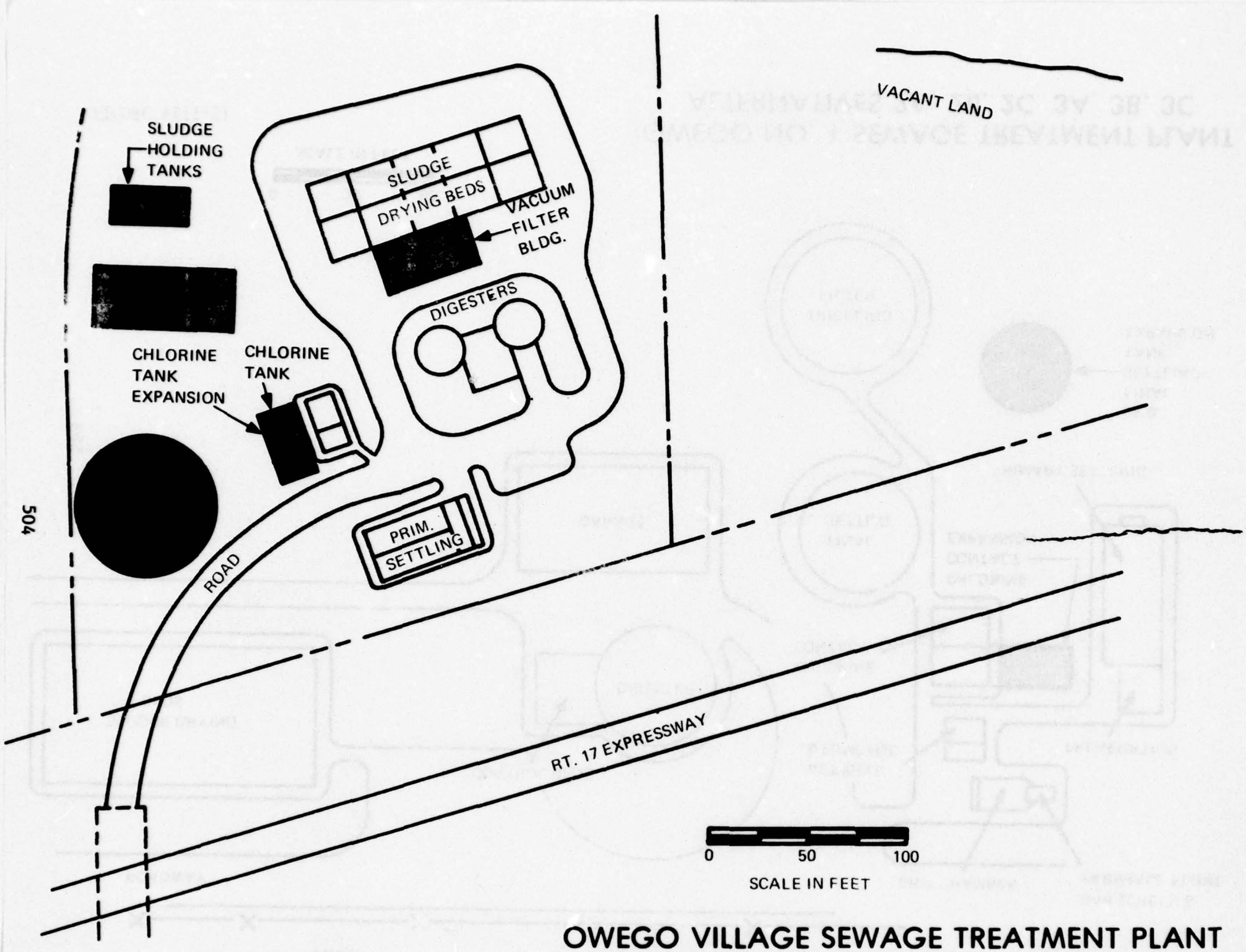


FIGURE VIII-52

OWEGO VILLAGE SEWAGE TREATMENT PLANT
ALTERNATIVES 2A, 2B, 2C, 3A, 3B, 3C

TABLE VIII-66

PLAN 3C

CONSTRUCTION SCHEDULE

	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[1] ($ Million)	Description
Binghamton-Johnson City	3.0	1977	1.21		Infiltration control
	39.5	1977	3.58		Storm overflow
	29.0	1977	0.05		Raw wastewater pumping
	21.5	1977	0.71		Aerator and clarifier
	22.3	1994	3.79		Nitrification
	24.5	2000	0.71		Aerator and clarifier
				7.1	
Endicott	0.8	1977	1.57		Nanticoke Valley Interceptor
	9.2	1983	1.95		Additional Secondary Treatment Capacity
				2.9	
East Owego	3.0	1992	1.26		General Expansion
				0.5	
West Owego	0.7	2000	0.47		General Expansion
				0.1	
Owego Village	1.0	1977	1.02		New Secondary treatment capacity
	3.0	1977	0.42		Micro-screening
	–	1977	0.58		Inflow control
				2.0	
Chenango Valley	1.0	1977	1.3		New Secondary treatment plant
	2.2	1985	1.2		General Expansion
				2.2	
Total Present Worth				14.8	

[1] 50 years @ 6-1/8%

TABLE VIII-67

CAPITAL COST BREAKDOWN FOR AN AERATION TANK AND SECONDARY CLARIFIER SET AT THE BINGHAMTON–JOHNSON CITY STP*

1)	Clarifier	$190,000
2)	Aeration Tank (30′ X 100′)	225,000
3)	Yard Piping & Renovation Instrumentation	80,000
4)	Additional Pumping and Valves	50,000
	Sub Total	545,000
	Eng. & Cont. @ 30%	164,000
	Total	$709,000

*July 1975 ENR 2248

TABLE VIII-58

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE SECONDARY TREATMENT
FACILITIES AT ENDICOTT STP FROM 7.7
TO 9.2 MGD CAPACITY*

1)	Preliminary Treatment	44,000
2)	Raw Wastes Pumps	74,000
3)	Trickling Filter (75′ϕ - 6′deep)	105,000
4)	Secondary Clarifier (50′ϕ X B′- 6 SWD)	158,000
5)	Chlorine Tank & Feed	51,000
6)	Digester Incl Bld Co. (42′ϕ - 30″high)	440,000
7)	Sludge Pumping	84,000
8)	Piles (for all structures)	345,000
9)	Misc. Electricity, Heating and Ventilation	148,000
10)	Yard Piping	54,000
	Subtotal	1,503,000
	Engineering & Contingencies @ 30%	451,000
	Total	1,954,000

*July 1975 ENR 2248.

TABLE VIII-69

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 2 CAPACITY FROM 2.0 MGD
TO 3 MGD*

1)	Prelimination Treatment & R.W. Pumps, wet well	137,000
2)	Preliminary Settling Tanks 1 tank – 1250 SF	63,000
3)	Aeration Tank & Air System 31,000 CF	116,000
4)	Pumps – Recirculating Piping, Structure Upgrade	80,000
5)	Sludge Thickener (for 4 MGD 24′ Diameter)	42,000
6)	Digester 45′ ϕ X 20′ Diameter	220,000
7)	Chlorine Tank & Feed System	58,000
8)	Sludge Pumping	47,000
9)	Final Clarifiers 55′ X 12′ X 8.5′ depth	74,000
10)	Misc. – Electricity, Heating and Ventilation Sitework	132,000
	Subtotal	969,000
	30% Engr & Cont	291,000
	Total	1,260,000

*July 1975 ENR 2248.

TABLE VIII-72

CAPITAL COST BREAKDOWN
FOR EXPANSION OF THE TOWN OF OWEGO
STP NO. 1 CAPACITY FROM 0.5 MGD
TO 0.7 MGD*

1)	Preliminary Treatment	26,000
2)	R.W. Pumps, Internal Piping	90,000
3)	Final Settling Tank (35′ϕ, Dept = 8′)	74,000
4)	Chlorination Tank (16′ X 40′ X 6′)	47,000
5)	Misc. Electrical Instrumentation Heating and Vent.	58,000
6)	Yard Piping, Effluent Piping	68,000
	Subtotal	363,000
	30% Engr. & Cont.	109,000
	Total	472,000

*July 1975 ENR 2248.

TABLE VIII-71

**CAPITAL COST BREAKDOWN
FOR UPGRADING THE VILLAGE OF OWEGO
STP SECONDARY TREATMENT CAPACITY
TO 1 MGD***

1)	Secondary Pump Station (Max Flow = 3.0)	200,000
2)	Trickling Filter A = 4,370 SF 75'ϕ X 5'-6" Stone Depth	87,000
3)	Final Settling TKS A = 1,904 SF 2 - 14' X 68' X 7' SWD	78,000
4)	Chlorine Tanks Vol = 3,800 16' X 40' X 6' SWD	25,000
5)	Sludge Holding Tanks 2 - 19' X 22' X 9' SWD With Mixers	60,000
6)	Vacuum Filter Area = 100 SF including Building 8'ϕ X 4' Face	160,000
7)	Preliminary Treatment – Screens, Grit removal Conninutor, etc.	65,000
8)	Site Work – Added Fill, Yard Piping, Grading, etc.	110,000
	Subtotal	785,000
	30% Engr. & Cont.	235,000
	Total	1,020,000

*July 1975 ENR 2248.

TABLE VIII-72

CAPITAL COST BREAKDOWN FOR SECONDARY TREATMENT PLANT
AT CHENANGO VALLEY
(Capacity of 1.0 mgd)*

1.	Preliminary Treatment		$ 74,000
2.	Primary Settling Tanks (A = 1250 SF 2 - 12' x 52' x 8' SWD)		74,000
3.	Aeration Tanks (V = 33,700 CF 2 - 19' x 74' x 12' SWD)		85,000
4.	Diffused Aeration System		139,000
5.	Secondary Settling Tanks (A = 1430 SF 2 - 13 x 55 x 10' SWD)		79,000
6.	Sludge Digestion (V - 40,000 CF 2 - 35ϕ)		262,000
7.	Chlorine Contact Tanks (V = 3480 CF 2 - 8' x 31' x 7' SWD)		24,000
8.	Chlorine Feed Equipment		28,000
9.	Yardwork		120,000
10.	Outfall to Chenango River		115,000
		Subtotal	$1,000,000
		Engr. & Cont. @ 30%	300,000
		Total	$1,300,000

*July 1975 ENR 2248

TABLE VIII-72 (Continued)

(Expansion of STP Capacity from 1.0 mgd to 2.2 mgd)*

1.	Preliminary Treatment	$ 88,000
2.	Primary Settling Tanks (A = 1560 SF 2 - 15' x 52' x 8' SWD)	82,000
3.	Aeration Tanks (V = 40,800 SF 2 - 23' x 74' x 12' SWD)	99,000
4.	Diffused Aeration System	177,000
5.	Secondary Settling Tanks (A - 1760 SF 2 - 16' x 55' x 10' SWD)	87,000
6.	Sludge Digestion (V = 44,000 CF 1 - 54'ϕ)	277,000
7.	Chlorine Contact Tanks (V = 4340 CF 2 - 10' x 31' x 7' SWD)	27,000
8.	Chlorine Feed Equipment	31,000
9.	Yardwork	55,000
	Subtotal	$923,000
	Engr. & Cont. @ 30%	277,000
	TOTAL	$1,200,000

*July 1975 ENR 2248

TABLE VIII-73

CAPITAL COST BREAKDOWN
FOR THE NANTICOKE INTERCEPTOR
(AVERAGE DESIGN FLOW = 0.8 M.G.D.*
PEAK DESIGN FLOW = 2.3 M.G.D.)

1)	18" sewer – 25,000' @ $38/LF	957,000
2)	110 manholes @ $1,100	120,000
3)	Misc. items	33,000
4)	Railroad crossing 27" casing 100' @ $270	27,000
5)	Creek Crossing 200/LF @ $350/LF	70,000
	Subtotal	1,207,000
	30% Engr. & Contr.	362,000
	Total	1,569,000

*use 18" ϕ sewer @ S = 0.0012 min. assume avg. depth = 12-14 feet.

July 1975 ENR 2248.

TABLE VIII-74

PLAN 3C
OPERATION AND MAINTENANCE COSTS

	First Year	Last Year	Description	Annual Cost ($ Million/Yr)	Present Worth[1] ($ Million)
Binghamton-Johnson City	1977	2026	Secondary	0.72	
	1977	2026	Storm Overflow	0.05	
	1994	2026	Nitrification	0.17	
					12.8
Endicott	1977	1982	Secondary	0.31	
	1983	2026	Secondary	0.36	
					5.1
Chenango	1977	1981	Secondary	0.086	
	1982	2026	Secondary	0.15	
					2.1
East Owego	1977	1991	Secondary	0.14	
	1992	2026	Secondary	0.19	
					2.4
West Owego	1977	1999	Secondary	0.04	
	2000	2026	Secondary	0.05	
					0.6
Owego Village	1977	2026	Secondary	0.07	
	1977	2026	Microscreening	0.002	
					1.1
				Total Present Worth:	24.1

[1] 50 years @ 6-1/8%.

TABLE VIII-75

PLAN 3C

CAPITAL COSTS (MILLION $) AND OPERATION AND MAINTENANCE COSTS ($\$10^3$/Yr)

Year	Binghamton Johnson Capital	Binghamton Johnson O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	E. Owego Capital	E. Owego O&M	W. Owego Capital	W. Owego O&M	Owego Village Capital	Owego Village O&M
1977	5.55	770	1.3	86	1.57	310		140		40	2.0	70
1982			1.2	150								
1983					1.95	360						
1985												
1989												
1990												
1992							1.26	190				
1994	3.79	940										
2000	.71								.47	50		
2010												
2018												
2026		940		150		360		190		50		70
Present Worth Million $	7.1	12.8	2.2	2.1	2.9	5.1	.5	2.4	.1	.6	2.0	1.1
Present Worth of Replacement Million $	6.4		.5		3.8		.8		.2		.5	
Total Present Worth Million $	26.3		4.8		11.8		3.7		0.9		3.6	

TABLE VIII-76

PLAN 3C – PRESENT WORTH[1] (MILLION $'s)

	Chenango Valley	Binghamton-Johnson City	Endicott	East Owego	West Owego	Owego Village	Total
Wastewater Treatment							
Capital	1.7	2.3	1.3	0.5	0.1	1.0	6.9
O & M	1.5	8.2	4.4	1.9	0.4	0.7	17.1
Replacement	0.4	5.2	3.6	0.7	0.1	0.3	10.3
TOTAL	3.6	15.7	9.3	3.1	0.6	2.0	34.3
Interceptors							
Capital	–	–	1.6	–	–	–	1.6
O & M	–	–	–	–	–	–	–
Replacement	–	–	–	–	–	–	–
TOTAL	–	–	1.6	–	–	–	1.6
Storm Overflow							
Capital	–	3.6	–	–	–	1.0	4.6
O & M	–	0.8	–	–	–	0.1	0.9
Replacement	–	0.9	–	–	–	0.1	1.0
TOTAL	–	5.3	–	–	–	1.2	6.5
Infiltration Control							
Capital	–	1.2	–	–	–	–	1.2
O & M	–	–	–	–	–	–	–
Replacement	–	–	–	–	–	–	–
TOTAL	–	1.2	–	–	–	–	1.2
Sludge							
Capital	0.465	0.05	0.01	0.005	0.001	0.002	0.5
O & M	0.6	3.8	0.7	0.5	0.2	0.3	6.1
Replacement	0.1	0.3	0.2	0.1	0.1	0.1	0.9
TOTAL	1.2	4.1	0.9	0.6	0.3	0.4	7.5
Totals							
Capital	2.2	7.1	2.9	0.5	0.1	2.0	14.8
O & M	2.1	12.8	5.1	2.4	0.6	1.1	24.1
Replacement	0.5	6.4	3.8	0.8	0.2	0.5	12.2
TOTAL	4.8	26.3	11.8	3.7	0.9	3.6	51.1

[1] Based on a 50 year economic life @ 6-1/8% and ENR = 2248.

PLAN 4

This plan provides for the application of Best Practicable Waste Treatment Technology (BPWTT) by 1983 and Advanced Waste Treatment Technology (AWT) by 1985 at all municipal wastewater treatment plants to achieve the Corps of Engineers definition of "zero discharge" and to maintain minimum average DO levels above 6 mg/l during low flow conditions. Storm overflow management facilities would be provided to maintain a minimum instantaneous DO level above 5.0 mg/l in the Susquehanna River during most storm overflow conditions.

MUNICIPAL WASTEWATER MANAGEMENT

Regionalization of STP's

The Broome County plants would be the existing Binghamton-Johnson City and Endicott STP's. Tioga County STP's would be the Town of Owego STP No. 1 (West Owego) and No. 2 (East Owego). The selection of this regionalization scheme was based on the economics of scale of AWT systems.

Service areas and regional interceptors included in this plan are shown in Plate 3. A regional interceptor would transmit Chenango Valley wastewater to be treated at the Binghamton-Johnson STP. Another regional interceptor would carry wastewater from the Village of Owego for treatment at the Town of Owego STP No. 1.

Treatment Levels and Processes

The same treatment levels would be applied at all the STP's. By 1983, BPWTT would provide nitrification of secondary effluent while AWT (1985) would provide denitrification, phosphorus removal, filtration, and carbon adsorption processes. The selection and design of the treatment processes is given in Chapter V.

Infiltration Control Level in the City of Binghamton

For the advanced waste treatment level and the regionalization scheme assumed in this alternative, an infiltration flow reduction of 3 mgd was found economically justifiable.

Nonstructural Measures for Flow Reduction

In this plan, the Village of Endicott and the Town of Owego would institute metered user rates. The increased price of water to the consumer should result in no increase in current per capita water consumption (and, therefore, per capita wastewater flows) in any of the service areas.

An educational program to encourage the use of water saving devices could result in sufficient reduction to maintain wastewater flows at their current per capita rate. Formal educational activities are detailed in the Specialty Appendix. This program would have the following major features:

1. Orient the program toward residences;

2. Stress the dollar savings of reduced water, sewer, and water heating bills to individual users by installing water saving devices;

3. Differentiate between homeowners, renters, and landlords;

4. Disseminate information by mail once a year;

5. Operate for 10-15 years;

6. Coordinate with local hardware suppliers to ensure the availability of water saving devices;

7. Promote local building code amendments with maximum water use specifications.

STORM OVERFLOW MANAGEMENT

Microscreening units followed by chlorination would be provided near five overflow sites in the City of Binghamton (see Figure VIII-53). This system, together with advanced wastewater treatment, would maintain a minimum instantaneous DO above 5 mg/l during most storm conditions. A detailed description of the overflow treatment system is given in Chapter VII. An estimate was also made of the cost of inflow control and microscreening treatment for the Village of Owego sewer system.

SLUDGE MANAGEMENT

Several sludge management techniques have been analyzed in Chapter VI. The land application of liquid sludge is recommended with landfill of dewatered sludge provided as a backup.

PERFORMANCE

Impact on the Susquehanna River dissolved oxygen levels:

1. Between 6 and 7 mg/l minimum daily average DO levels during low river flow and dry weather wastewater flow conditions.

2. Between 5 and 6 mg/l minimum instantaneous DO levels during design storm overflow conditions.

The pollutant mass loads discharged to the Susquehanna and Chenango Rivers by the STP's proposed in this plan are summarized in Table VIII-77.

CONSTRUCTION SCHEDULE

Nitrification facilities will be added to all the treatment plants included in this plan by 1983. At the Binghamton-Johnson City STP, the nitrification tanks will have a capacity of 20.5 mgd which will be sufficient to treat the future wastewater flows throughout the 50-year planning period. At the Village of Endicott STP, the nitrification facility will have a capacity of 7.5 mgd which will be sufficient to treat wastewater flows to the year 2027. At the Town of Owego STP No. 2, the nitrification facility will have an initial capacity of 2.0 mgd, which will be expanded to 2.5 mgd in the year 2000. At the Town of Owego STP No. 1, the nitrification units will have a capacity of 1.5 mgd which will be sufficient to the year 2027 (see Figures VIII-54 to VIII-57).

The Advanced Waste Treatment Units will be added to all the STP's by 1985. The capacity installed at the Binghamton-Johnson City STP, Endicott STP, Town of Owego STP No. 2 and the Town of Owego STP No. 1 will be 20.5, 7.5, 2.5, and 1.5 mgd, respectively, which will be sufficient to treat the wastewater flow projected up to the year 2027.

Expansion of the existing secondary wastewater facilities will be required to meet the increasing flow expected during the planning period.

At the Town of Owego STP No. 2, the plant capacity will be expanded from the present 2.0 mgd to 2.5 mgd by the year 2000. At the Town of Owego STP No. 1, the plant capacity will be expanded by 1977 from 0.5 mgd to 1.5 mgd, when the wastewater flow from the Village of Owego is connected to STP No. 1.

At the Binghamton-Johnson City STP, the raw wastewater pumping capacity will be increased from 18.3 mgd to 29 mgd by 1977. Also, two aerator and clarifier units will be added to increase the capacity from 18.3 mgd to 24.4 mgd by 1977.

At Endicott, no expansion of secondary facilities is required due to the small increases in flow expected during the planning period.

The Chenango service area wastewater will be transmitted to the City of Binghamton wastewater collection system through a 16-inch force main that has a design flow of 2.2 mgd. A 1.38 million gallon holding tank would be used to equalize the wastewater flow and so reduce the pump station and the force main costs. This interceptor will be built by 1977.

The Nanticoke service area wastewater will be transmitted to the Endicott STP through a 25,000 foot long, 18-inch diameter sewer that has a design average flow of 0.8 mgd and a peak flow of 2.3 mgd. This interceptor will be built by 1977.

Storm overflow control facilities are assumed to be constructed at the City of Binghamton by 1977. Five microscreening facilities having a total treatment capacity of 39.5 mgd will be located as shown in Figure VIII-53.

At the Village of Owego, a microscreening unit with a capacity of 3 mgd would be built by 1977 to provide treatment of storm overflows. Also, steps will be taken to control direct stream inflow into the sewer system.

A summary of the construction schedule of wastewater treatment facilities needed at each service area is given in Table VIII-78, and the facilities are shown on Figures VIII-58 through VIII-61. As can be seen, this installation will create space problems at the Binghamton-Johnson City STP, and may even be impractical on the existing site. This is a function of the level of treatment in this plan, not the degree of regionalization.

COST ANALYSIS

The capital cost breakdown for the expansion and/or upgrading of the existing treatment facilities at the Binghamton-Johnson City STP, the Town of Owego STP No. 1, and the Town of Owego STP No. 2, are shown in Tables VIII-79 through VIII-81, respectively. Tables VIII-82, VIII-83, and VIII-84 break down the capital costs for the Chenango Valley, Nanticoke Valley, and Owego Village interceptors, respectively. The capital costs for the AWT processes are presented in Table VIII-85.

The O&M costs for each service area during the planning period are given in Table VIII-86.

The capital, O&M, and replacement costs for each service area are summarized in Table VIII-87. Also shown in the table are the years these expenditures will be incurred. These correspond to the construction schedule in the previous section. The replacement costs were estimated using an equivalent annual sinking fund approach.

The present worth of all the expenditures during the 50-year planning period is given in Table VIII-88 for each service area. The present worth costs for this plan are: capital -- $27.5 million, O&M -- $48.4 million, and replacement -- $14.3 million, for a total cost of $90.2 million.

SUMMARY OF COSTS

A comparison of the capital, operation and maintenance and replacement costs of each plan is presented in Table VIII-89. Differences among the three regionalization schemes (labelled A, B, and C) for Plans 2 and 3 are not significant. By adding nitrification at Binghamton-Johnson City, Plan 3 would be somewhat more expensive than Plan 2; increases of approximately 60 percent and 50 percent, respectively over the total cost of the Baseline Plan. The cost of providing AWT in Plan 4 would be almost three times the total Baseline Cost.

CITY OF BINGHAMTON

LOCATION OF PLANNED OVERFLOW CONTROL FACILITIES

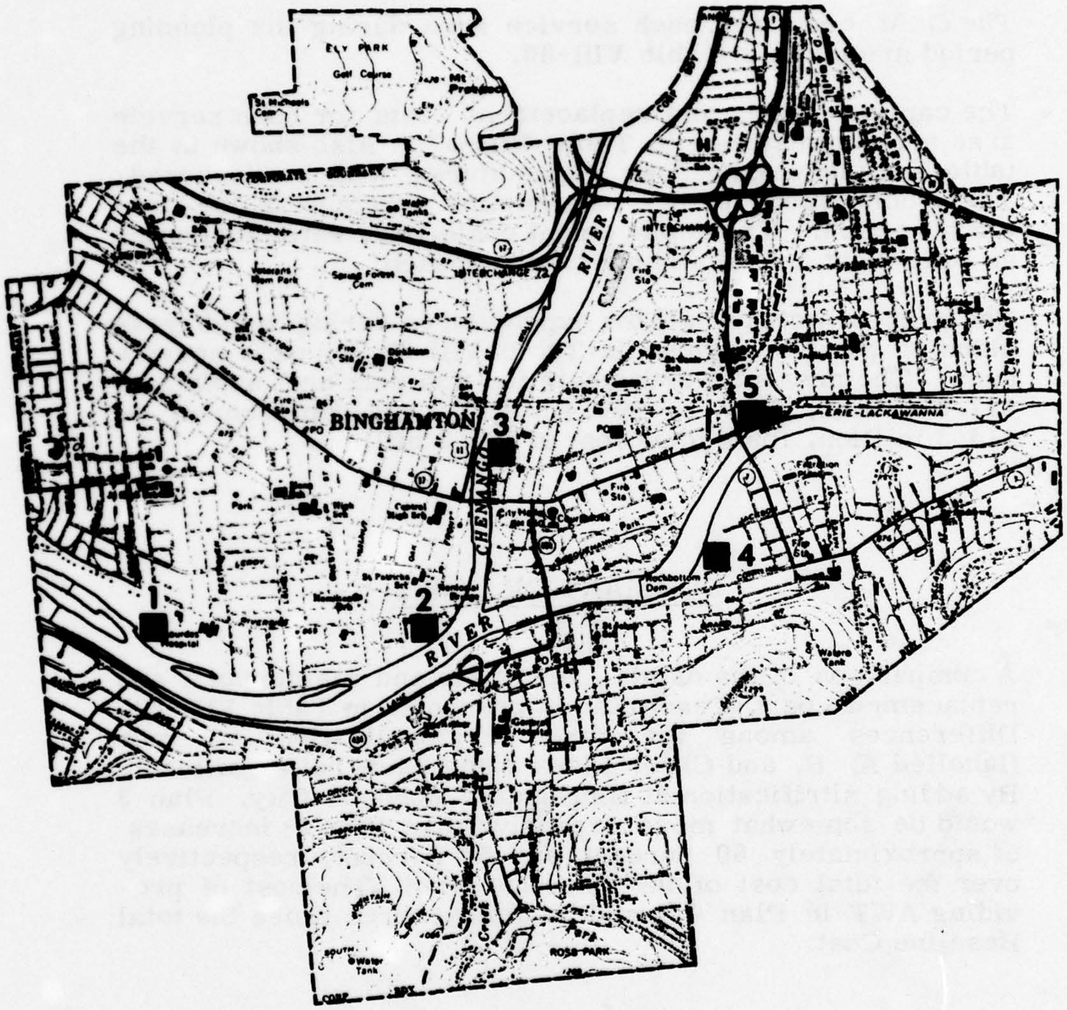


FIGURE VIII-53

TABLE VIII-77

PLAN 4
AVERAGE DAILY POLLUTION LOADS TO THE SUSQUEHANNA RIVER[1]

Source	Parameter					
	Flow (M.G.D)	SS lb/day	BOD lb/day	NOD lb/day	N lb/day	P lb/day
Binghamton-Johnson	20.4	350	350	400	350	85
Endicott	7.5	150	150	175	150	40
Chenango						
Owego #2	2.3	45	45	55	45	10
Owego #1	1.5	40	40	45	40	10
Owego Village						
Total	31.7	585	585	675	585	145

[1]Year 2020

BINGHAMTON - JOHNSON CITY STP EXPANSIONS, BIO AWT

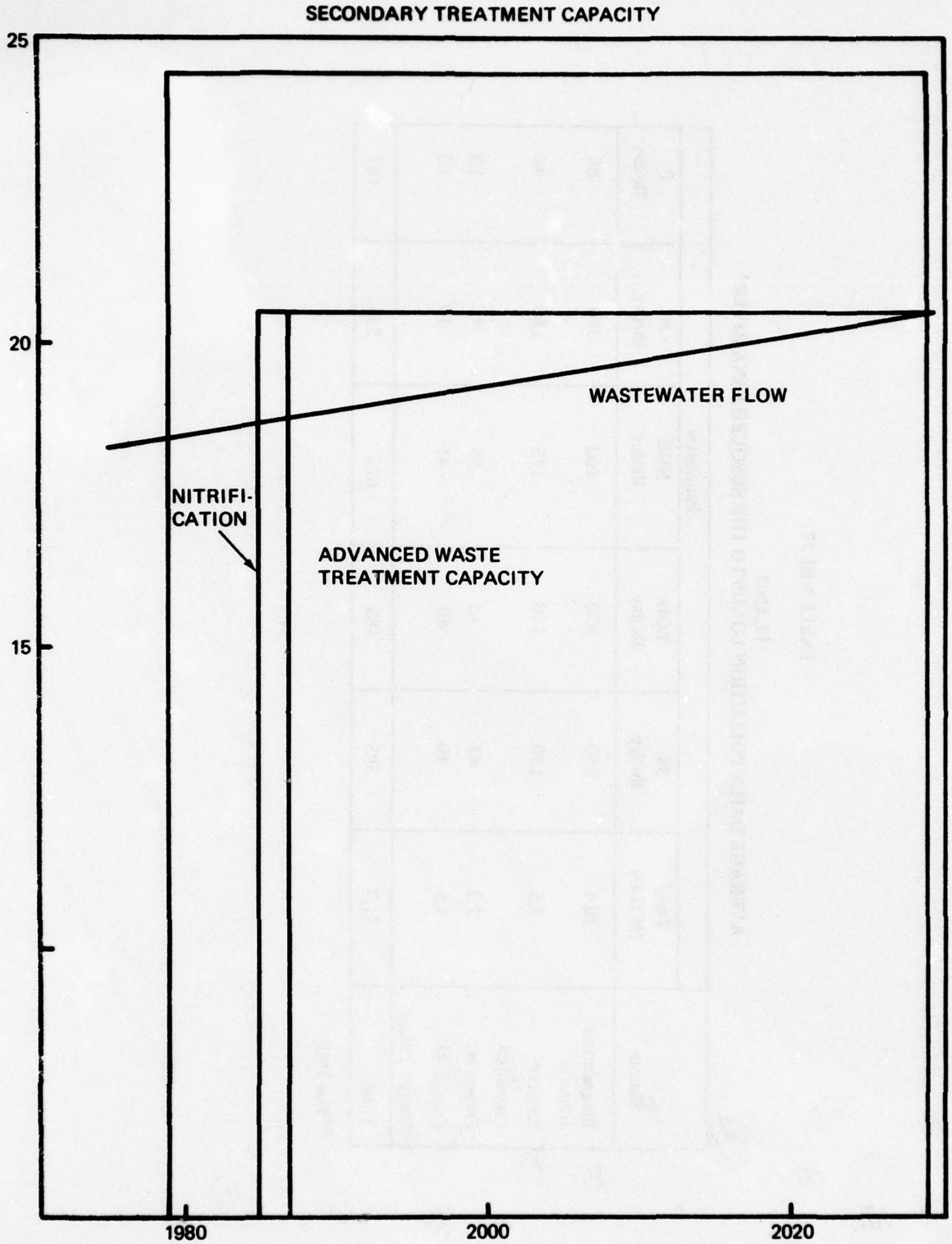


Figure VIII-54

ENDICOTT STP EXPANSIONS, BIO AWT

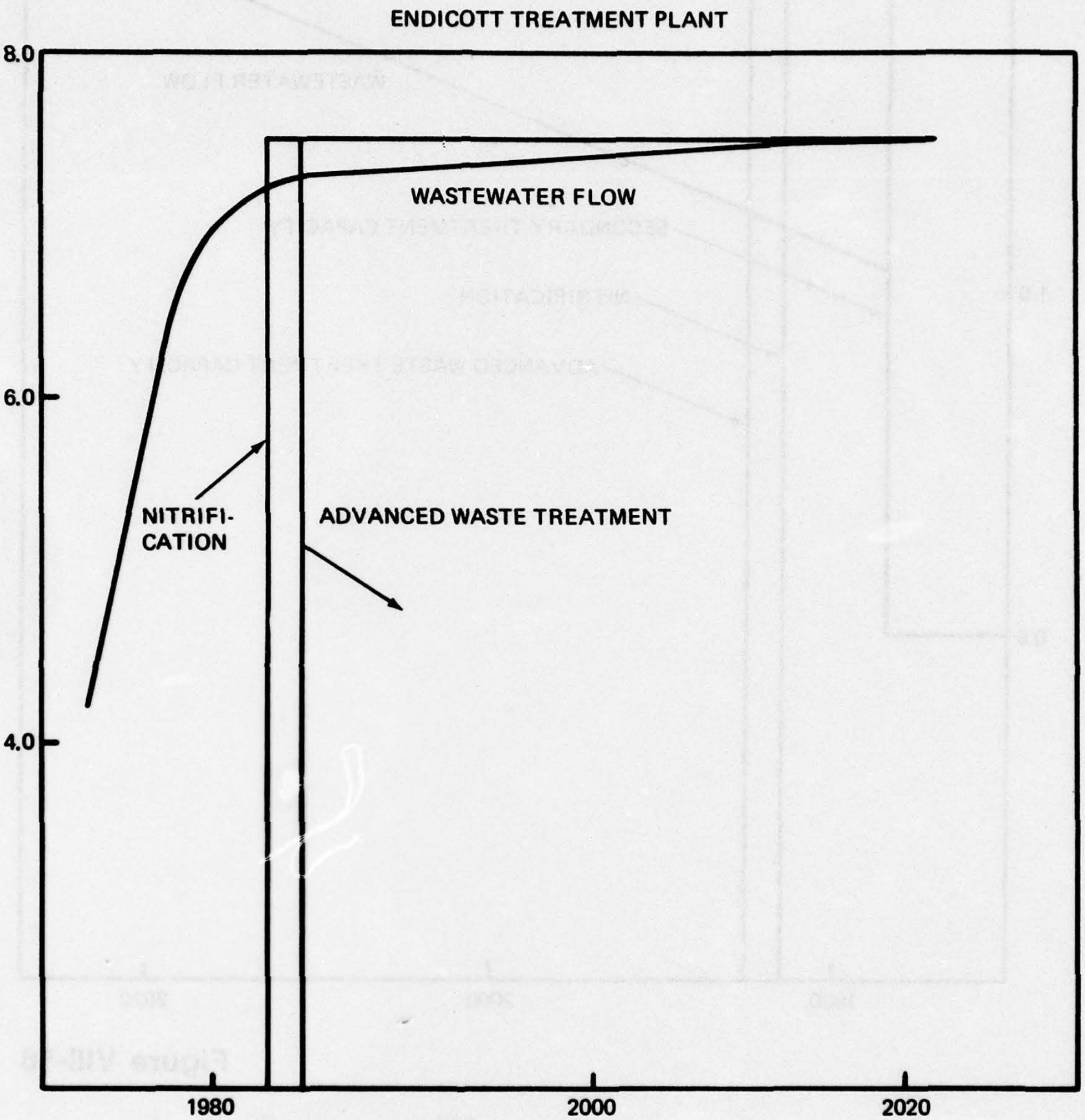


FIGURE 4-3 ENDICOTT STP BIO–AWT

Figure VIII-55

OWEGO STP NO. 1 EXPANSIONS, BIO AWT

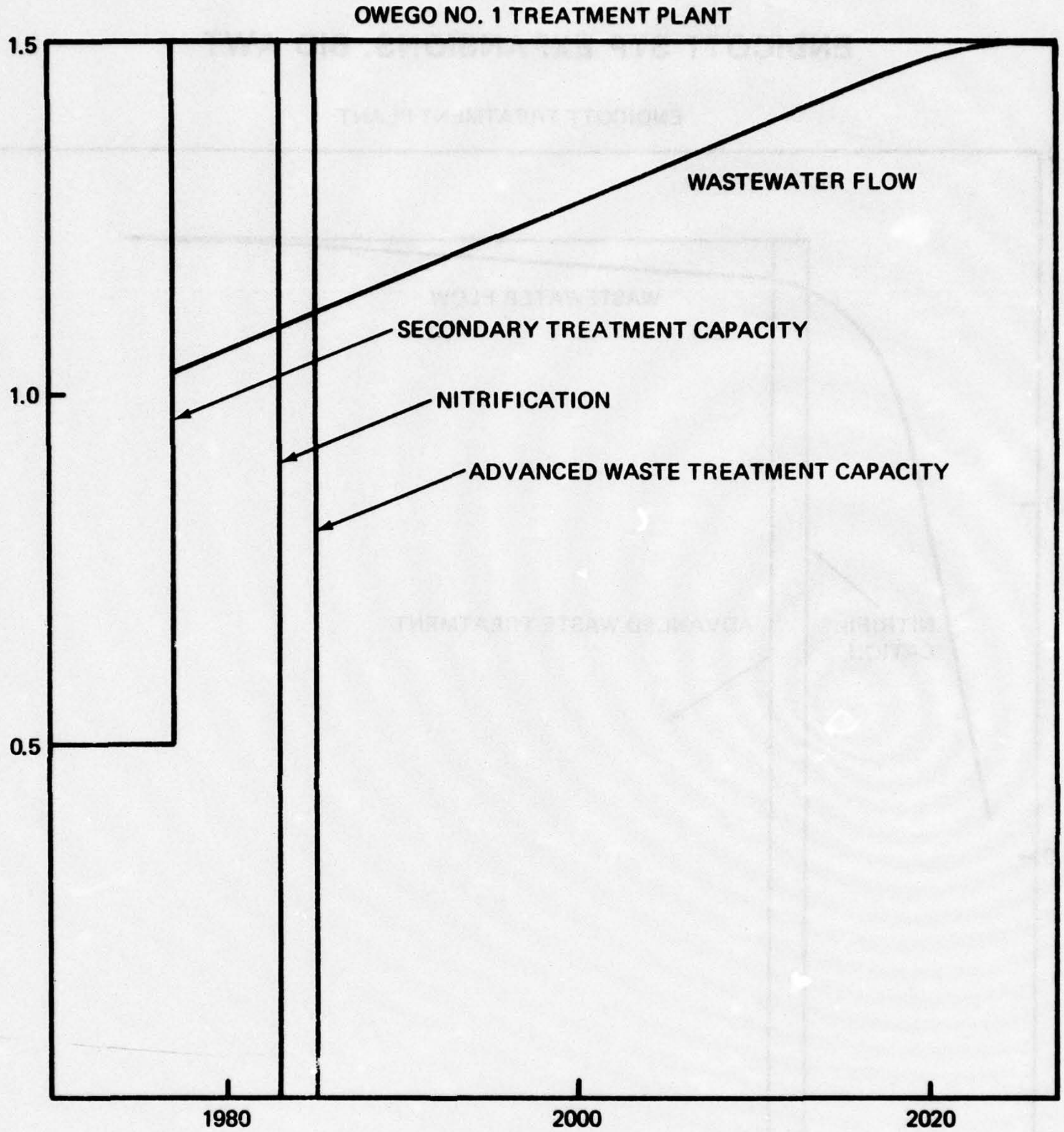


Figure VIII-56

OWEGO STP NO. 2 EXPANSIONS, BIO AWT

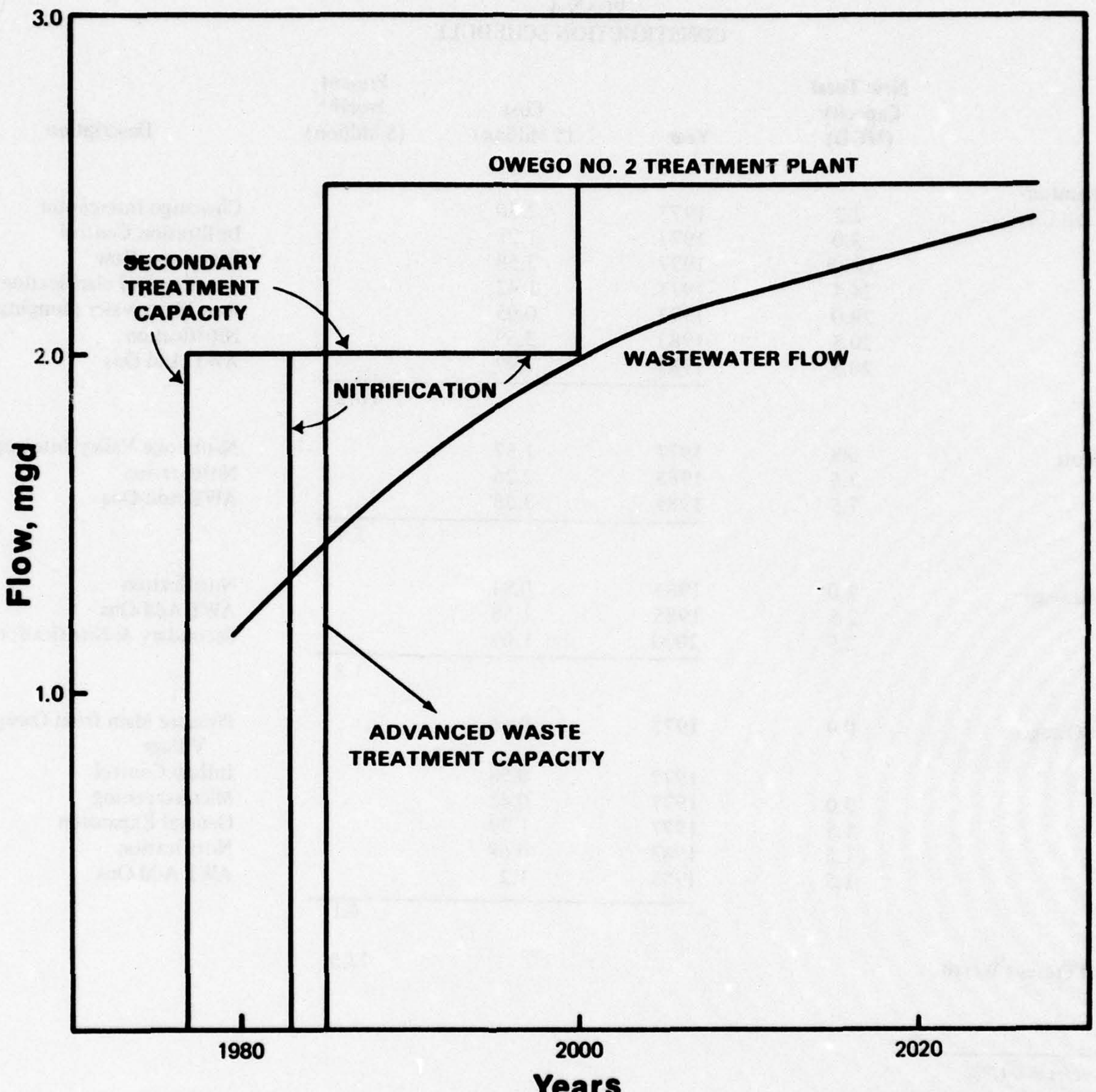


Figure VIII-57

TABLE VIII-78

PLAN 4
CONSTRUCTION SCHEDULE

	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[1] ($ Million)	Description
Binghamton-Johnson City	2.2	1977	2.50		Chenango Interceptor
	3.0	1977	1.21		Infiltration Control
	39.5	1977	3.58		Storm overflow
	24.4	1977	1.42		Aeration and clarification
	29.0	1977	0.05		Raw Wastewater pumping
	20.5	1983	3.69		Nitrification
	20.5	1985	7.99		AWT Add-Ons
				16.2	
Endicott	0.8	1977	1.57		Nanticoke Valley Interceptor
	7.5	1983	2.26		Nitrification
	7.5	1985	3.58		AWT Add-Ons
				5.4	
East Owego	2.0	1983	0.84		Nitrification
	2.5	1985	1.58		AWT Add-Ons
	2.5	2000	1.05		Secondary & Nitrification
				1.8	
West Owego	0.9	1977	0.66		Pressure Main from Owego Village
		1977	0.58		Inflow Control
	3.0	1977	0.42		Microscreening
	1.5	1977	1.29		General Expansion
	1.5	1983	0.68		Nitrification
	1.5	1985	1.2		AWT Add-Ons
				4.1	
Total Present Worth				27.5	

[1] 50 years @ 6-1/8%

BINGHAMTON - JOHNSON CITY STP, PLAN 4

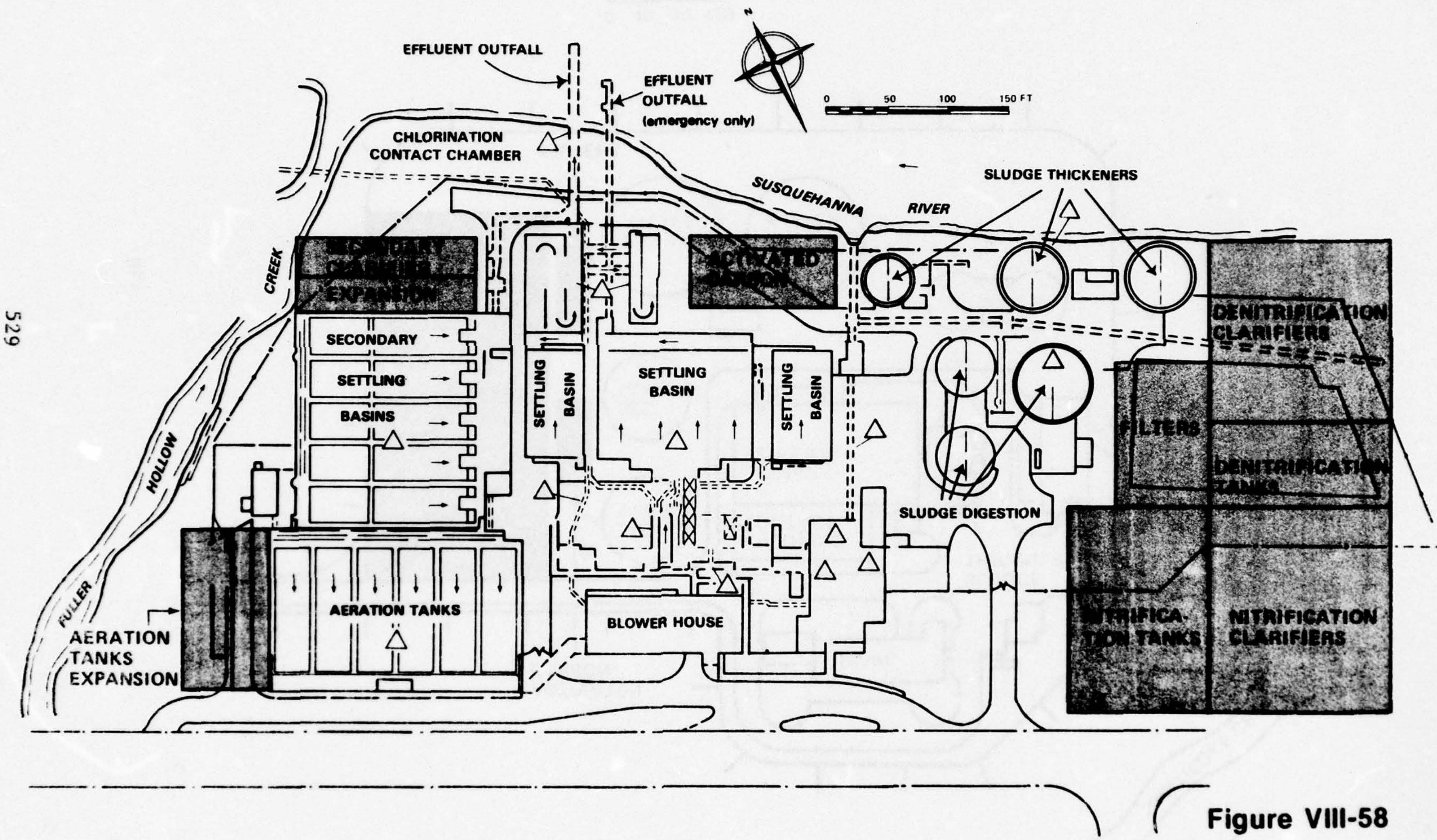


Figure VIII-58

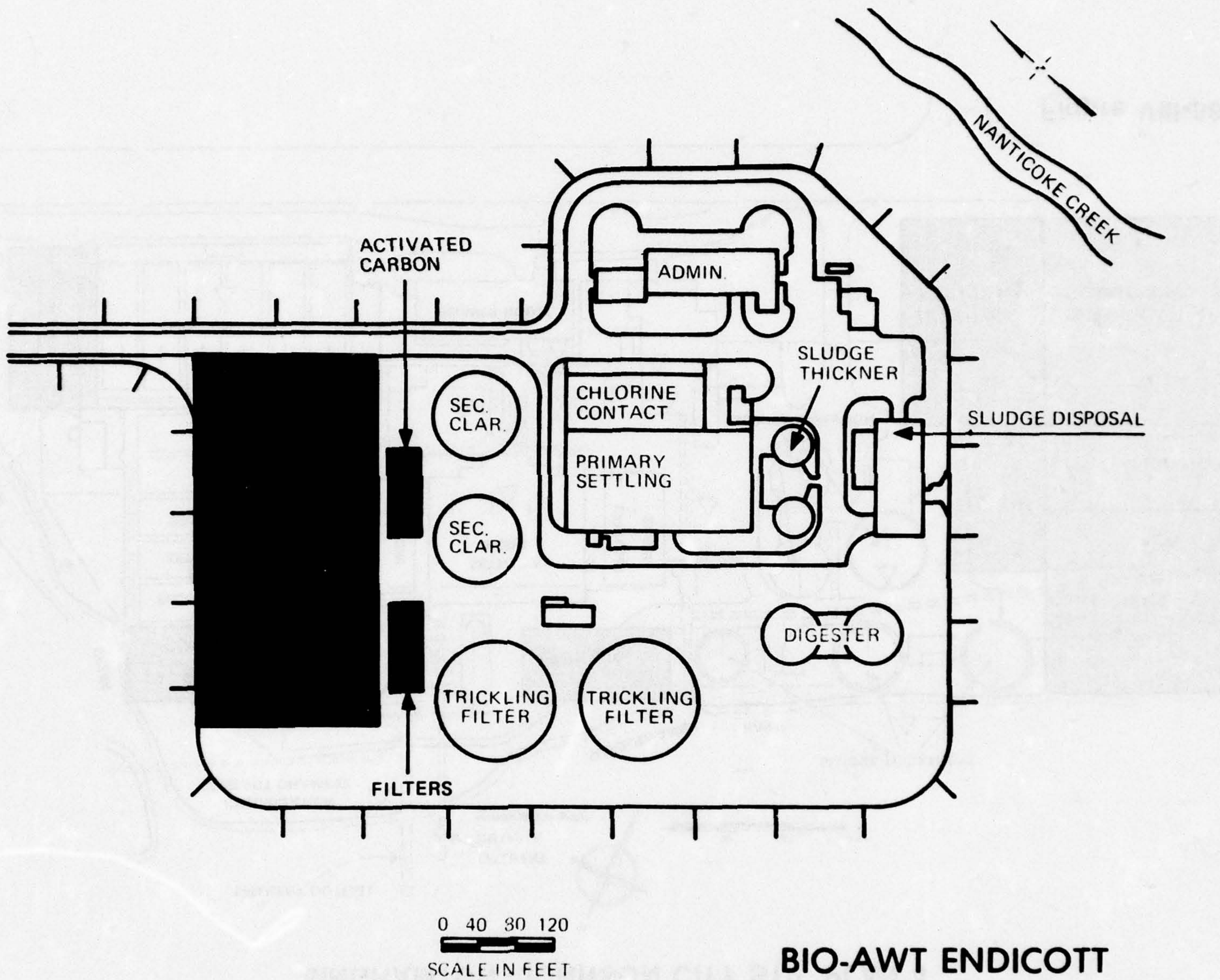


FIGURE VIII-59

BIO-AWT ENDICOTT SEWAGE TREATMENT PLANT ALTERNATIVE 4

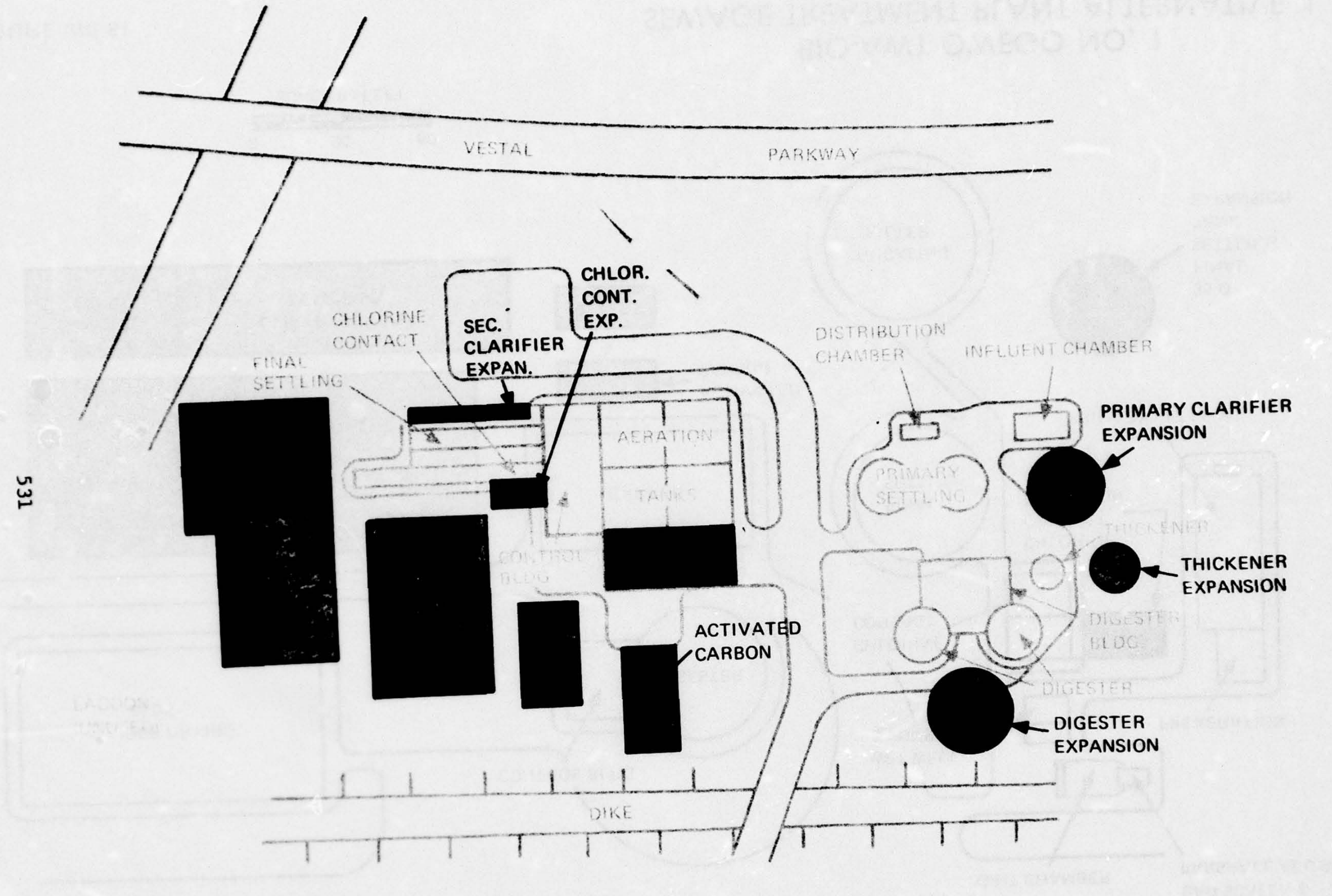


FIGURE VIII-60

BIO-AWT OWEGO NO. 2
SEWAGE TREATMENT PLANT ALTERNATIVE 4

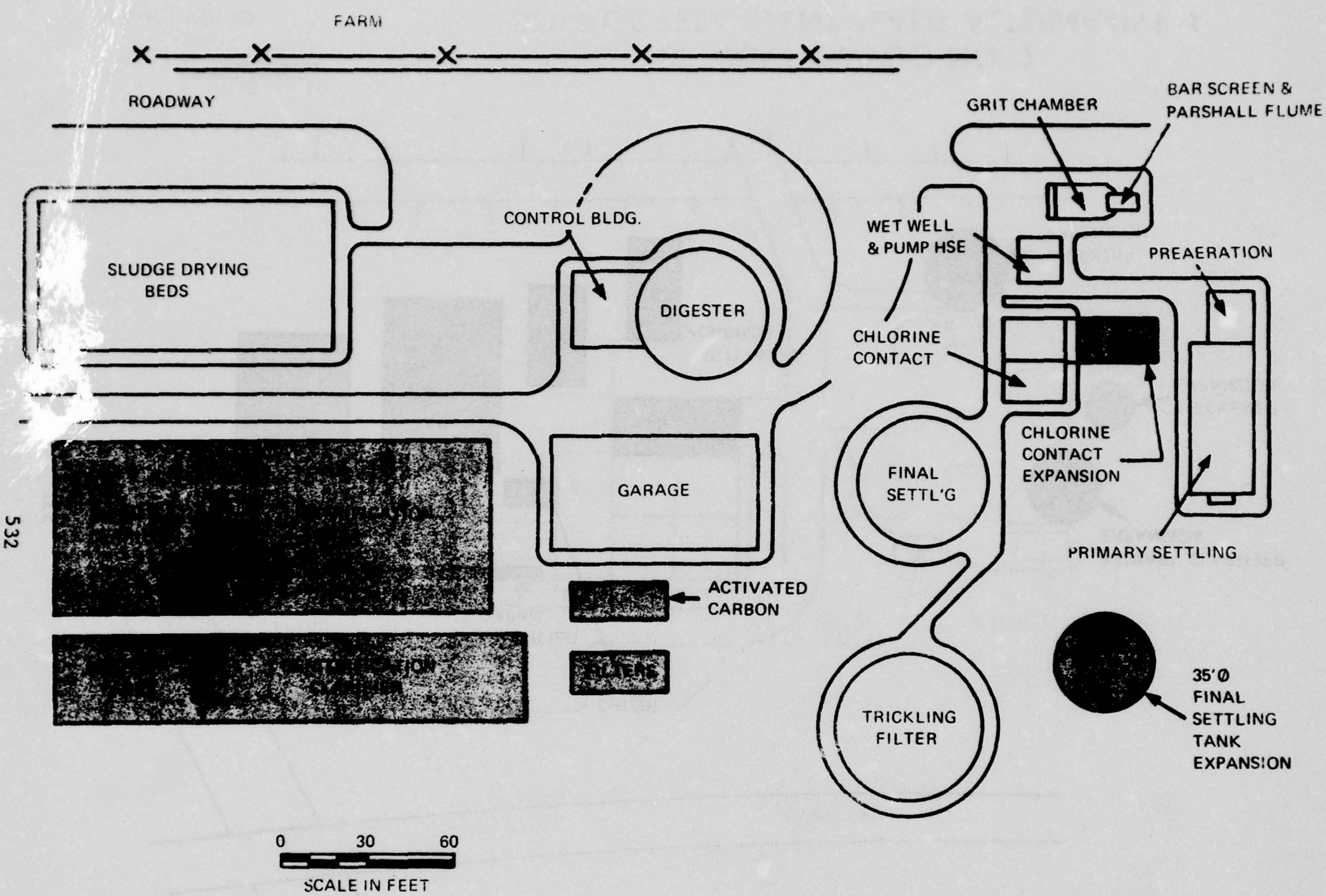


FIGURE VIII-61

BIO-AWT OWEGO NO. 1
SEWAGE TREATMENT PLANT ALTERNATIVE 4

TABLE VIII-79

CAPITAL COST BREAKDOWN FOR AN AERATION TANK AND SECONDARY CLARIFIER SET AT THE BINGHAMTON–JOHNSON CITY STP*

1)	Clarifier	$190,000
2)	Aeration Tank (30′ × 100′)	225,000
3)	Yard Piping & Renovation Instrumentation	80,000
4)	Additional Pumping and Valves	50,000
	Sub Total	545,000
	Eng. & Cont. @ 30%	164,000
	Total	$709,000

*July 1975 ENR 2248

TABLE VIII-80

CAPITAL COST BREAKDOWN FOR THE EXPANSION OF THE TOWN OF OWEGO STP NO. 2 CAPACITY FROM 2.0 MGD TO 2.5 MGD*

1)	Preliminary Work	$ 55,000
2)	Primary Settling	60,000
3)	Aeration Tank & Air System	80,000
4)	Recycle Pumps	40,000
5)	Sludge Thickener	40,000
6)	Chlorine Tank	35,000
7)	Final Clarifier	70,000
8)	Site Work – Electrical and piping	60,000
		440,000
	Eng & Cont. At 30%	132,000
	TOTAL	572,000

*July 1975 ENR 2248

TABLE VIII-81

PLAN 4

TOWN OF OWEGO STP NO 1
COST TO EXPAND SECONDARY CAPACITY TO 1.5 MGD*

a.	prelim. treatment	$ 40,000
b.	raw wastewater pumps – pump building	115,000
c.	primary settling tanks	70,000
d.	digestors	155,000
e.	final settling tanks	75,000
f.	chlorination tank	20,000
g.	administration – lab	55,000
h.	vacuum filters	200,000
i.	sitework – yard piping	85,000
j.	electrical – instrumentation	70,000
k.	trickling filter	55,000
		$940,000
	30% engineering and contingencies	280,000
	TOTAL	$1,220,000

*July 1975 ENR 2248

TABLE VIII-82

CAPITAL COST BREAKDOWN
FOR THE CHENANGO INTERCEPTOR
(AVERAGE DESIGN FLOW = 2.2 M.G.D.
MAXIMUM DESIGN FLOW = 3.3 M.G.D. FOR 10 HOURS)*

1)	Pump Station Q = 2.2 M.G.D. (Aug.)	265,000
2)	Holding Basin with Aerators (125' x 125' = 1.375 MG)	
	a) Concrete	515,000
	b) Excavation	53,000
	c) Aerators	160,000
	d) Piling	145,000
	e) Piping Instr. Values, etc.	180,000
	Subtotal	1,053,000
3)	Dewatering, Sheeting, Paving, etc.	53,000
4)	Force Main, Q = 2.2 M.G.D. 16" – 13,500' @ $33.70/LF	455,000
5)	Highway Crossing, 30" casing – 250' @ $264/LF	66,000
6)	Land	32,000
	Subtotal	1,924,000
	30% Engr. & Contr.	577,000
	Total	2,501,000

*July 1975 ENR 2248.

TABLE VIII-83

CAPITAL COST BREAKDOWN
FOR THE NANTICOKE INTERCEPTOR
(AVERAGE DESIGN FLOW = 0.8 M.G.D.*
PEAK DESIGN FLOW = 2.3 M.G.D.)

1)	18" sewer – 25,000' @ $38/LF	957,000
2)	110 manholes @ $1,100	120,000
3)	Misc. items	33,000
4)	Railroad crossing 27" casing 100' @ $270	27,000
5)	Creek Crossing 200/LF @ $350/LF	70,000
	Subtotal	1,207,000
	30% Engr. & Contr.	362,000
	Total	1,569,000

*use 18" ϕ sewer @ S = 0.0012 min. assume avg. depth = 12-14 feet.

July 1975 ENR 2248.

TABLE VIII-84

CAPITAL COST BREAKDOWN FOR THE
OWEGO VILLAGE INTERCEPTOR[1]

1.	Pump Station modifications at Owego Village STP	$ 60,000
2.	River Crossing – No change to existing pipe	0
3.	Pump Station modification at north side of river	35,000
4.	Force Main from river station to 5th Avenue Pump Station	180,000
5.	Modifications to 5th Avenue Pump Station	55,000
6.	Force Main from 5th Avenue to Owego STP #1	180,000
	Subtotal	$510,000
	30% Engr. & Contin.	150,000
	Total	$660,000

[1] Owego Village STP to Owego STP #1; 0.9 mgd capacity; July 1975 ENR 2248

TABLE VIII-85

PLAN 4
BREAKDOWN OF CAPITAL COSTS OF ADVANCED TREATMENT ADD-ONS

Service Area	Component	Cost ($ Million)
Binghamton-Johnson City	Denitrification	2.4
	Phosphorus Removal	0.15
	Rapid Filtration	1.6
	Activated Carbon	3.9
	TOTAL	8.0
Endicott	Denitrification	1.0
	Phosphorus Removal	0.06
	Rapid Filtration	0.8
	Activated Carbon	1.7
	TOTAL	3.6
East Owego	Denitrification	0.4
	Phosphorus Removal	0.03
	Rapid Filtration	0.4
	Activated Carbon	0.8
	TOTAL	1.6
	Nitrification (2000)	*0.14
West Owego	Denitrification	0.3
	Phosphorus Removal	0.03
	Rapid Filtration	0.27
	Activated Carbon	0.6
	TOTAL	1.2
	TOTAL COST (1985)	14.4

* East Owego Nitrification Added in Two Stages

TABLE VIII-86

PLAN 4
OPERATION AND MAINTENANCE COSTS

	First Year	Last Year	Description	Annual Cost ($ Million/yr)	Present Worth[1] ($ Million)
Binghamton-	1977	2026	Storm Overflow	0.05	
Johnson City	1977	2026	Secondary	0.78	
	1983	2026	Nitrification	0.17	
	1985	2026	AWT Add-Ons	1.47	
					28.3
Endicott	1977	2026	Secondary	0.31	
	1983	2026	Nitrification	0.09	
	1985	2026	AWT Add-Ons	0.61	
					11.4
Chenango					
East Owego	1977	1999	Secondary	0.14	
	1983	1999	Nitrification	0.04	
	1985	2026	AWT Add-Ons	0.26	
	2000	2026	Secondary + Nitrif.	0.21	
					5.0
West Owego	1977	2026	Microscreening	0.002	
	1977	2026	Secondary	0.09	
	1983	2026	Nitrification	0.04	
	1985	2026	AWT Add-Ons	0.18	
					3.7
				Total Present Worth	48.4

[1] 50 years @ 6-1/8%.

TABLE VIII-87

PLAN 4

CAPITAL COSTS (MILLION \$) AND OPERATION AND MAINTENANCE COSTS ($\$10^3$/Yr)

Year	Binghamton Johnson Capital	Binghamton Johnson O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	E. Owego Capital	E. Owego O&M	W. Owego Capital	W. Owego O&M	Owego Village Capital	Owego Village O&M
1977	8.76	830			1.57	310		140	2.95	92		
1983	3.69	1000			2.26	400	.84	180	.68	132		
1985	7.90	2470			3.58	1010	1.58	440	1.2	312		
1989												
1990												
1992												
1995												
2000							1.05	650				
2010												
2018						—						
2027		2470				1010		650		312		
Present Worth Million \$	16.2	28.3			5.4	11.4	1.8	5.0	4.1	3.7		
Present Worth of Replacement Million \$	8.0				4.4		1.1		.8			
Total Present Worth Million \$	52.5				21.2		7.9		8.6			

TABLE VIII-88

PLAN 4 COSTS – PRESENT WORTH[1]
(MILLION $'s)

	Binghamton-Johnson City	Endicott	East Owego	West Owego	TOTAL
Wastewater Management					
Capital	8.8	3.8	1.8	2.4	16.8
O & M	21.5	10.0	4.3	3.1	38.9
Replacement	6.8	4.2	1.0	0.6	12.6
TOTAL	37.1	18.0	7.1	6.1	68.3
Interceptors					
Capital	2.5	1.6	-	0.7	4.8
O & M	-	-	-	-	-
Replacement	-	-	-	-	-
TOTAL	2.5	1.6	-	0.7	4.8
Storm Overflows					
Capital	3.6	-	-	1.0	4.6
O & M	0.8	-	-	0.1	0.9
Replacement	0.9	-	-	0.1	1.0
TOTAL	5.3	-	-	1.2	6.5
Infiltration Control					
Capital	1.2	-	-	-	1.2
O & M	-	-	-	-	-
Replacement	-	-	-	-	-
TOTAL	1.2	-	-	-	1.2
Sludge					
Capital	0.08	0.02	0.01	0.01	0.1
O & M	6.0	1.4	0.7	0.5	8.6
Replacement	0.3	0.2	0.1	0.1	0.7
TOTAL	6.4	1.6	0.8	0.6	9.4
Totals					
Capital	16.2	5.4	1.8	4.1	27.5
O & M	28.3	11.4	5.0	3.7	48.4
Replacement	8.0	4.4	1.1	0.8	14.3
TOTAL PLAN COST	52.5	21.2	7.9	8.6	90.2

[1]Based on a 50 year economic life @ 6 1/8% and ENR = 2248

TABLE VIII-89

FINAL PLANS COST SUMMARY

(Million $'s)

Cost*	1	2A	2B	2C	3A	3B	3C	4
PRESENT WORTH								
Capital	2.6	13.5	12.6	12.6	15.8	14.8	14.8	27.5
O&M	19.5	22.1	23.2	23.2	22.9	24.1	24.1	48.4
Replacement	9.8	11.6	11.9	11.9	11.8	12.2	12.2	14.3
TOTAL PRESENT WORTH	31.9	47.2	47.7	47.7	50.5	51.1	51.1	90.2
AVERAGE ANNUAL COST	2.1	3.0	3.1	3.1	3.3	3.3	3.3	5.8

*6 1/8 percent interest and 50-year project life.

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ADDENDUM A

SHORT TERM (20 YEAR) ECONOMIC ANALYSIS

PURPOSE

The costs presented in the main body of this Appendix were based on a 50-year economic life. In accordance with recent EPA guidelines (Guidelines on Preparing a Facilities Plan, May 1975) and Corps of Engineers' regulations (ER 1105-2-22 and ER 1105-2-180), this addendum presents the costs of the final plans projected over a 20-year economic life. A comparison is then made between this short term (20 year) and long term (50 year) analyses and their effect on the relative costs of the four Plans for Choice.

METHODOLOGY

To obtain the costs for the 20-year planning period, the construction and O&M schedules developed for the 50-year analysis (Chapter VIII) were used for the years 1977 through 1996. As replacement costs for the long term analysis were based on a 27-year composite useful life (see Chapter II), no replacement was assumed for the short term analysis. Costs are expressed as both present worth and average annual, computed at an interest rate of 6 1/8 percent.

COSTS

The following tables present the actual capital and O&M costs and the years in which these would occur during the 20-year planning period. Also given, for comparison purposes, are the present worth and average annual costs of each plan.

Table A-1

CAPITAL COSTS ($ Million) AND OPERATION & MAINTENANCE COSTS ($ Thousands)

PLAN 1

Year	BINGHAMTON-JOHNSON CITY Capital	BINGHAMTON-JOHNSON CITY O&M	CHENANGO Capital	CHENANGO O&M	ENDICOTT Capital	ENDICOTT O&M	EAST OWEGO Capital	EAST OWEGO O&M	WEST OWEGO Capital	WEST OWEGO O&M	OWEGO VILLAGE Capital	OWEGO VILLAGE O&M
1977		721			1.57	310		140		40	1.02	70
1982												
1983												
1985												
1991												
1992												
1994												
1996		721				310		140		40		70
Present Worth[1] ($ Million)		8.2			1.57	3.5		1.6		0.5	1.02	0.8
Total Present Worth[1] ($ Million)	8.2				5.1		1.6		0.5		1.8	

1. 20 years at 6 1/8%.

TABLE A-2

PRESENT WORTH AND AVERAGE ANNUAL COSTS[1]

($ Million)

PLAN 1

Service Area	Present Worth Capital	Present Worth O&M	Present Worth Total	Average Annual Cost Capital	Average Annual Cost O&M	Average Annual Cost Total
Binghamton--Johnson City	--	8.2	8.2	--	0.72	0.72
Chenango	--	--	--	--	--	--
Endicott	1.57	3.5	5.1	0.14	0.31	0.45
East Owego	--	1.6	1.6	--	0.14	0.14
West Owego	--	0.45	0.45	--	0.04	0.04
Owego Village	1.02	0.8	1.8	0.09	0.07	0.16
TOTAL	2.59	14.55	17.1	0.23	1.28	1.51

1. 20 years at 6 1/8%.

Table A-3

CAPITAL COSTS ($ Million) AND OPERATION & MAINTENANCE COSTS ($ Thousands)

PLAN 2A

Year	BINGHAMTON-JOHNSON CITY Capital	BINGHAMTON-JOHNSON CITY O&M	CHENANGO Capital	CHENANGO O&M	ENDICOTT Capital	ENDICOTT O&M	EAST OWEGO Capital	EAST OWEGO O&M	WEST OWEGO Capital	WEST OWEGO O&M	OWEGO VILLAGE Capital	OWEGO VILLAGE O&M
1977	8.23	830			1.57	310		140		40	2.0	70
1982						310						
1983					1.95	360						
1985												
1991								140				
1992							1.26	190				
1994												
1996		830				360		190		40		70
Present Worth[1] ($ Million)	8.23	9.4			2.93	3.84	0.5	1.7		0.45	2.0	0.8
Total Present Worth[1] ($ Million)	17.6				6.8		2.2		0.45		2.8	

1. 20 years at 6 1/8%.

TABLE A-4

PRESENT WORTH AND AVERAGE ANNUAL COSTS[1]

($ Million)

PLAN 2A

Service Area	Present Worth Capital	O&M	Total	Average Annual Cost Capital	O&M	Total
Binghamton-- Johnson City	8.23	9.42	17.6	0.72	0.83	1.55
Chenango	--	--	--	--	--	--
Endicott	2.93	3.84	6.8	0.26	0.34	0.60
East Owego	0.5	1.7	2.2	0.04	0.15	0.19
West Owego	--	0.45	0.45	--	0.04	0.04
Owego Village	2.0	0.8	2.8	0.18	0.07	0.25
TOTAL	13.7	16.2	29.9	1.20	1.43	2.63

1. 20 years at 6 1/8%.

Table A-5

CAPITAL COSTS ($ Million) AND OPERATION & MAINTENANCE COSTS ($ Thousands)

PLAN 2B

Year	BINGHAMTON-JOHNSON CITY Capital	BINGHAMTON-JOHNSON CITY O&M	CHENANGO Capital	CHENANGO O&M	ENDICOTT Capital	ENDICOTT O&M	EAST OWEGO Capital	EAST OWEGO O&M	WEST OWEGO Capital	WEST OWEGO O&M	OWEGO VILLAGE Capital	OWEGO VILLAGE O&M
1977	4.55	770	1.9	130	1.57	310		140		40	2.0	70
1982						310						
1983				130	1.95	360						
1985			0.46	150								
1991	0.71							140				
1992							1.26	190				
1994												
1996		770		150		360		190		40		70
Present Worth[1] ($ Million)	4.86	8.74	2.2	1.6	2.93	3.84	0.5	1.7		0.45	2.0	0.8
Total Present Worth[1] ($ Million)	13.6		3.8		6.8		2.2		0.45		2.8	

1. 20 years at 6 1/8%.

TABLE A-6

PRESENT WORTH AND AVERAGE ANNUAL COSTS[1]

($ Million)

PLAN 2B

Service Area	Present Worth Capital	Present Worth O&M	Present Worth Total	Average Annual Cost Capital	Average Annual Cost O&M	Average Annual Cost Total
Binghamton--Johnson City	4.86	8.74	13.6	0.43	0.77	1.20
Chenango	2.2	1.6	3.8	0.19	0.14	0.33
Endicott	2.93	3.84	6.8	0.26	0.34	0.60
East Owego	0.5	1.7	2.2	0.04	0.15	0.19
West Owego	--	0.45	0.45	--	0.04	0.04
Owego Village	2.0	0.8	2.8	0.18	0.07	0.25
TOTAL	12.5	17.1	29.6	1.10	1.51	2.61

1. 20 years at 6 1/8%.

Table A-7

CAPITAL COSTS ($ Million) AND OPERATION & MAINTENANCE COSTS ($ Thousands)

PLAN 2C

Year	BINGHAMTON-JOHNSON CITY Capital	BINGHAMTON-JOHNSON CITY O&M	CHENANGO Capital	CHENANGO O&M	ENDICOTT Capital	ENDICOTT O&M	EAST OWEGO Capital	EAST OWEGO O&M	WEST OWEGO Capital	WEST OWEGO O&M	OWEGO VILLAGE Capital	OWEGO VILLAGE O&M
1977	4.55	770	1.3	86	1.57	310		140		40	2.0	70
1982			1.2	150		310						
1983					1.95	360						
1985												
1991	0.71							140				
1992							1.26	190				
1994												
1996		770		150		360		190		40		70
Present Worth[1] ($ Million)	4.86	8.74	2.2	1.4	2.93	3.84	0.5	1.7		0.45	2.0	0.8
Total Present Worth[1] ($ Million)	13.6		3.6		6.8		2.2		0.45		2.8	

1. 20 years at 6 1/8%.

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TABLE A-8

PRESENT WORTH AND AVERAGE ANNUAL COSTS[1]

($ Million)

PLAN 2C

Service Area	Present Worth Capital	Present Worth O&M	Present Worth Total	Average Annual Cost Capital	Average Annual Cost O&M	Average Annual Cost Total
Binghamton-- Johnson City	4.86	8.74	13.6	0.43	0.77	1.20
Chenango	2.2	1.4	3.6	0.19	0.12	0.32
Endicott	2.93	3.84	6.8	0.26	0.34	0.60
East Owego	0.5	1.7	2.2	0.04	0.15	0.19
West Owego	--	0.45	0.45	--	0.04	0.04
Owego Village	2.0	0.8	2.8	0.18	0.07	0.25
TOTAL	12.5	16.9	29.4	1.10	1.49	2.59

1. 20 years at 6 1/8%.

Table A-9

CAPITAL COSTS ($ Million) AND OPERATION & MAINTENANCE COSTS ($ Thousands)

PLAN 3A

Year	BINGHAMTON-JOHNSON CITY		CHENANGO		ENDICOTT		EAST OWEGO		WEST OWEGO		OWEGO VILLAGE	
	Capital	O&M	Capital	O&M	Capital	O&M	Capital	O&M	Capital	O&M	Capital	O&M
1977	8.75	830			1.57	310		140		40	2.0	70
1982						310						
1983					1.95	360						
1985												
1991								140				
1992							1.26	190				
1994	4.05	1000										
1996		1000				360		190		40		70
Present Worth[1] ($ Million)	10.2	9.6			2.93	3.84	0.5	1.7		0.45	2.0	0.8
Total Present Worth[1] ($ Million)	19.8				6.8		2.2		0.45		2.8	

1. 20 years at 6 1/8%.

TABLE A-10

PRESENT WORTH AND AVERAGE ANNUAL COSTS[1]

($ Million)

PLAN 3A

Service Area	Present Worth			Average Annual Cost		
	Capital	O&M	Total	Capital	O&M	Total
Binghamton--Johnson City	10.22	9.6	19.8	0.90	0.84	1.74
Chenango	--	--	--	--	--	--
Endicott	2.93	3.84	6.8	0.26	0.34	0.60
East Owego	0.5	1.7	2.2	0.04	0.15	0.19
West Owego	--	0.45	0.45	--	0.04	0.04
Owego Village	2.0	0.8	2.8	0.18	0.07	0.25
TOTAL	15.7	16.3	32.0	1.38	1.44	2.82

1. 20 years at 6 1/8%.

Table A- 11

CAPITAL COSTS ($ Million) AND OPERATION & MAINTENANCE COSTS ($ Thousands)

PLAN 3B

Year	BINGHAMTON-JOHNSON CITY Capital	BINGHAMTON-JOHNSON CITY O&M	CHENANGO Capital	CHENANGO O&M	ENDICOTT Capital	ENDICOTT O&M	EAST OWEGO Capital	EAST OWEGO O&M	WEST OWEGO Capital	WEST OWEGO O&M	OWEGO VILLAGE Capital	OWEGO VILLAGE O&M
1977	5.55	770	1.9	130	1.57	310		140		40	2.0	70
1982						310						
1983				130	1.95	360						
1985			0.46	150								
1991								140				
1992		770					1.26	190				
1994	3.79	940										
1996		940		150		360		190		40		70
Present Worth[1] ($ Million)	6.9	8.9	2.2	1.6	2.93	3.84	0.5	1.7		0.45	2.0	0.8
Total Present Worth[1] ($ Million)	15.8		3.8		6.8		2.2		0.45		2.8	

1. 20 years at 6 1/8%.

TABLE A-12

PRESENT WORTH AND AVERAGE ANNUAL COSTS[1]

($ Million)

PLAN 3B

Service Area	Present Worth			Average Annual Cost		
	Capital	O&M	Total	Capital	O&M	Total
Binghamton--Johnson City	6.9	8.9	15.8	0.61	0.78	1.39
Chenango	2.2	1.6	3.8	0.19	0.14	0.33
Endicott	2.93	3.84	6.8	0.26	0.34	0.60
East Owego	0.5	1.7	2.2	0.04	0.15	0.19
West Owego	--	0.45	0.45	--	0.04	0.04
Owego Village	2.0	0.8	2.8	0.18	0.07	0.25
TOTAL	14.5	17.3	31.8	1.28	1.52	2.80

1. 20 years at 6 1/8%.

Table A-13

CAPITAL COSTS ($ Million) AND OPERATION & MAINTENANCE COSTS ($ Thousands)

PLAN 3C

Year	BINGHAMTON-JOHNSON CITY Capital	BINGHAMTON-JOHNSON CITY O&M	CHENANGO Capital	CHENANGO O&M	ENDICOTT Capital	ENDICOTT O&M	EAST OWEGO Capital	EAST OWEGO O&M	WEST OWEGO Capital	WEST OWEGO O&M	OWEGO VILLAGE Capital	OWEGO VILLAGE O&M
1977	5.55	770	1.3	86	1.57	310		140		40	2.0	70
1982			1.2	150		310						
1983					1.95	360						
1985												
1991								140				
1992		770					1.26	190				
1994	3.79	940										
1996		940		150		360		190		40		70
Present Worth[1] ($ Million)	6.9	8.9	2.2	1.4	2.93	3.84	0.5	1.7		0.45	2.0	0.8
Total Present Worth[1] ($ Million)	15.8		3.6		6.8		2.2		0.45		2.8	

1. 20 years at 6 1/8%.

TABLE A-14

PRESENT WORTH AND AVERAGE ANNUAL COSTS[1]

($ Million)

PLAN 3C

Service Area	Present Worth Capital	Present Worth O&M	Present Worth Total	Average Annual Cost Capital	Average Annual Cost O&M	Average Annual Cost Total
Binghamton-- Johnson City	6.9	8.9	15.8	0.61	0.78	1.39
Chenango	2.2	1.4	3.6	0.19	0.13	0.32
Endicott	2.93	3.84	6.8	0.26	0.34	0.60
East Owego	0.5	1.7	2.2	0.04	0.15	0.19
West Owego	--	0.45	0.45	--	0.04	0.04
Owego Village	2.0	0.8	2.8	0.18	0.07	0.25
TOTAL	14.6	17.1	31.7	1.28	1.51	2.79

1. 20 years at 6 1/8%.

Table A-15

CAPITAL COSTS ($ Million) AND OPERATION & MAINTENANCE COSTS ($ Thousands)

PLAN 4

Year	BINGHAMTON-JOHNSON CITY		CHENANGO		ENDICOTT		EAST OWEGO		WEST OWEGO		OWEGO VILLAGE	
	Capital	O&M	Capital	O&M	Capital	O&M	Capital	O&M	Capital	O&M	Capital	O&M
1977	8.76	830			1.57	310		140	2.95	92		
1982		830				310		140		92		
1983	3.69	1000			2.26	400	0.84	180	0.68	132		
1985	7.9	2470			3.58	1010	1.58	440	1.2	312		
1991												
1992												
1994												
1996		2470				1010		440		312		
Present Worth[1] ($ Million)	16.3	18.1			5.38	7.26	1.6	3.2	4.17	2.23		
Total Present Worth[1] ($ Million)	34.4				12.6		4.8		6.4			

1. 20 years at 6 1/8%.

TABLE A-16

PRESENT WORTH AND AVERAGE ANNUAL COSTS[1]

($ Million)

PLAN 4

Service Area	Present Worth Capital	Present Worth O&M	Present Worth Total	Average Annual Cost Capital	Average Annual Cost O&M	Average Annual Cost Total
Binghamton--Johnson City	16.25	18.13	34.4	1.43	1.60	3.03
Chenango	--	--	--	--	--	--
Endicott	5.38	7.26	12.6	0.47	0.64	1.11
East Owego	1.57	3.19	4.8	0.14	0.28	0.42
West Owego	4.17	2.23	6.4	0.37	0.20	0.57
Owego Village	--	--	--	--	--	--
TOTAL	27.4	30.8	58.2	2.41	2.71	5.12

1. 20 years at 6 1/8%.

CONCLUSION

Table A-17 and Figure A-1 provide a cost comparison of the final plans for both the 20-year and 50-year planning periods. The relative costs of Plans 1, 2, 3, and 4 does not significantly differ between the short term planning period and the long term. For Plans labelled A, B, and C, a small change in the relative ranking is noted, however. With a 20-year economic life, plans labelled "C" are the least expensive (but only slightly), followed by "B", and "A" plans. This finding supports the ISMG recommendation of Plan 2C as the Plan of Choice.

TABLE A-17

SUMMARY OF COSTS

20 YEAR AND 50 YEAR PROJECTIONS[1]

($ Million)

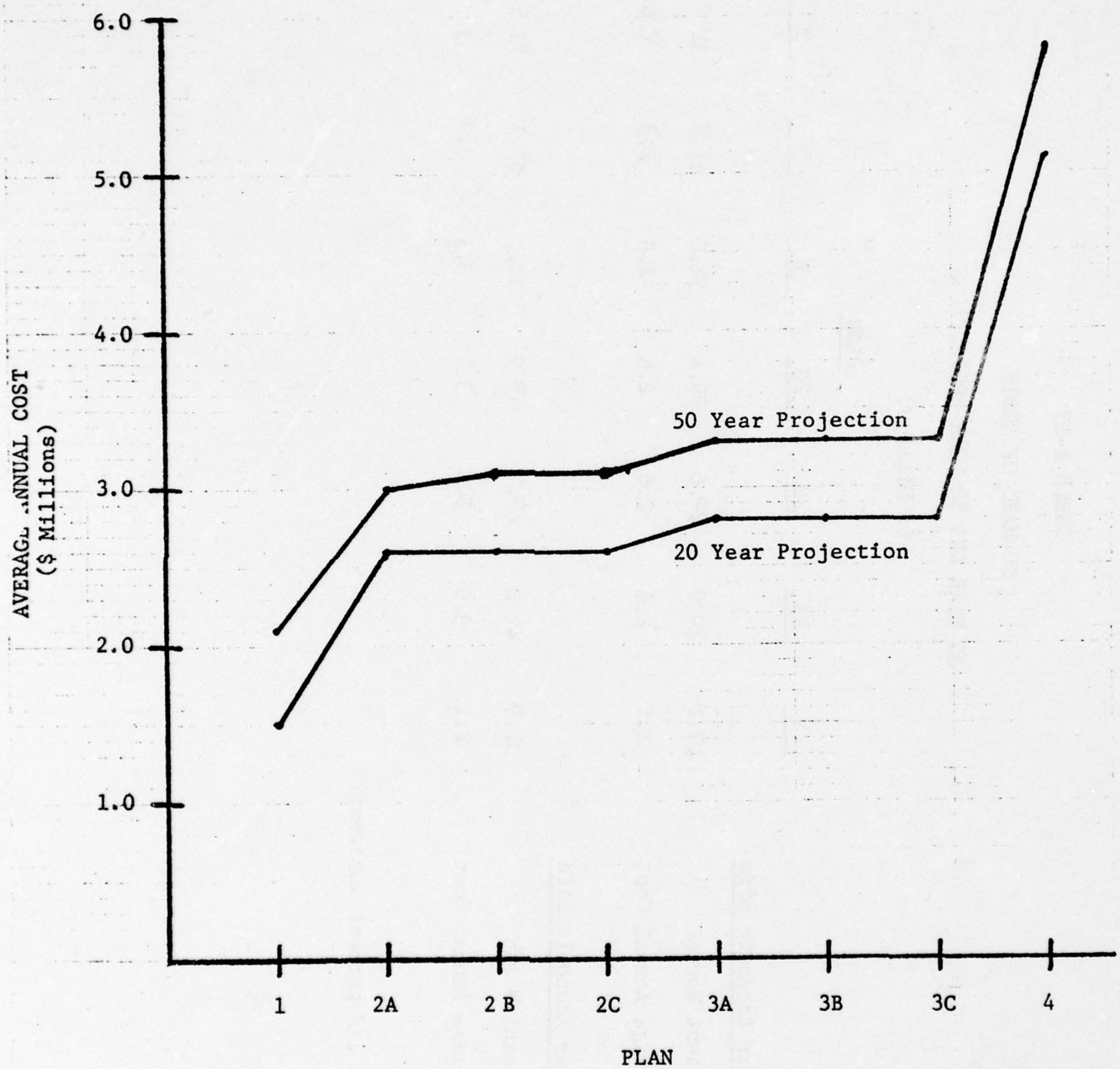
	PLAN							
	1	2A	2B	2C	3A	3B	3C	4
20-Year Economic Life								
Present Worth	17.1	29.9	29.6	29.4	32.0	31.8	31.7	58.2
Average Annual Cost	1.5	2.6	2.6	2.6	2.8	2.8	2.8	5.1
50-Year Economic Life								
Present Worth	31.9	47.2	47.7	47.7	50.5	51.1	51.1	90.2
Average Annual Cost	2.1	3.0	3.1	3.1	3.3	3.3	3.3	5.8

1. 6 1/8 percent interest.

FIGURE A - 1

SUMMARY OF COSTS

20 YEAR AND 50 YEAR PROJECTIONS



ADDENDUM B

COST SUMMARY WITH 50 PERCENT ENGINEERING AND CONTINGENCIES COST

PURPOSE

Throughout the main body of the Report for the Binghamton Wastewater Management Study, a 30 percent factor for engineering and contingencies was included in the capital costs. That is, the estimated capital cost was 130 percent of the estimated construction cost. This factor was used to cover the cost of engineering design services, administrative charges, and unforeseen expenditures.

Due to the complexity of the proposed wastewater management plans and the preliminary nature of the projects investigated, it was suggested during review of the draft Report that an engineering and contingencies factor of 50 percent also be investigated. As requested, the purpose of this addendum is to present cost information for each of the Plans for Choice with a 50 percent engineering and contingencies factor for construction costs.

METHODOLOGY

The cost analysis for the 50 percent engineering and contingencies factor is similar to cost analysis already described in Chapter VIII. Capital costs were estimated for each specific project according to the necessary construction items. The construction schedule for each service area within each plan was modified to reflect the new engineering and contingencies factor of 50 percent. These new data were then tabulated on a time-scaled table showing the appropriate year for each expenditure, including operation and maintenance and replacement costs (O&M and replacement costs were

assumed to be the same values already presented in Chapter VIII). A summary table was then prepared showing the present worth costs (capital, O&M and replacement) at each service area for plan components such as wastewater treatment, interceptors, storm overflows, infiltration control, and sludge management. Tables B-1 through B-23 summarize the appropriate data using the 50 percent engineering and contingencies factors.

SUMMARY AND COMPARISON

Table B-24 summarizes both the total present worth cost and the average annual cost for each plan with a 50 percent engineering and contingencies factor included. Table B-25 compares the total costs of plans developed in Chapter VIII including a 30 percent factor with the total costs of the same plans presented in this addendum including a 50 percent factor. The relative costs of Plans 1, 2, 3, and 4 were not altered by the higher factor for engineering and contingencies.

TABLE B-1

PLAN 1

CAPITAL COSTS (MILLION \$) AND OPERATION & MAINTENANCE COSTS (\$$10^3$/Yr) [1]
(Engineering & Contingencies @ 50%)

Year	Binghamton--Johnson City Capital	Binghamton--Johnson City O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	East Owego Capital	East Owego O&M	West Owego Capital	West Owego O&M	Owego Village Capital	Owego Village O&M
1977		721			1.81	310		140		40	1.17	70
1983												
1985												
1989												
1990												
1992												
1995												
2000												
2010												
2018												
2027		721				310		140		40		70
Present Worth (Million \$)		11.1			1.81	4.7		2.1		0.6	1.17	1.0
Present Worth of Replacement (Million \$)	5.0				3.5		0.7		0.2		0.4	
Total Present Worth (Million \$)	16.1				10.0		2.8		0.8		2.6	

1 Engineering and Contingencies @ 50%

TABLE B-2

PLAN 1 (BASELINE) COSTS
PRESENT WORTH (MILLION $'s) COSTS[1]

	Bing-JC	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment						
Capital	-	-	-	-	1.2	1.2
O & M	7.3	4.0	1.6	0.4	0.7	14.0
Replacement	4.7	3.3	0.6	0.1	0.3	9.0
TOTAL	12.0	7.3	2.2	0.5	2.2	24.2
Interceptors						
Capital	-	1.8	-	-	-	1.8
O & M	-	-	-	-	-	-
Replacement	-	-	-	-	-	-
TOTAL	-	1.8	-	-	-	1.8
Storm Overflows -- NONE						
Capital						
O & M						
Replacement						
TOTAL						
Infiltration Control -- NONE						
Capital						
O & M						
Replacement						
TOTAL						
Sludge						
Capital	-	-	-	-	-	-
O & M	3.8	0.7	0.5	0.2	0.3	5.5
Replacement	0.3	0.2	0.1	0.1	0.1	0.8
TOTAL	4.1	0.9	0.6	0.3	0.4	6.3
Totals						
Capital	-	1.8	-	-	1.2	3.0
O & M	11.1	4.7	2.1	0.6	1.0	19.5
Replacement	5.0	3.5	0.7	0.2	0.4	9.8
TOTAL PLAN COST	16.1	10.0	2.8	0.8	2.6	32.3

1 Based on 50 year economic life @ 6 1/8% and ENR = 2248.
Engineering and Contingencies @ 50%.

TABLE B-3

PLAN 2A CONSTRUCTION SCHEDULE

Location	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[2] ($ Million)	Description
Binghamton-	2.2	1977	2.88		Chenango Interceptor
Johnson City	1.0	1977	0.24		Infiltration Control
	39.5	1977	4.13		Storm Overflow
	29.0	1977	0.06		Raw Wastewater Pumping
	26.5	1977	1.64		Aerator and Clarifier (2 sets)
	29.0	1997	0.45		Primary Tank
	30.5	2025	0.82		Aeration and Clarifier (1 set)
				9.13	
Endicott	0.8	1977	1.81		Nanticoke Valley Interceptor
	9.2	1983	2.25		Addl. Secondary Treatment Capacity
				3.38	
East Owego (#2)	3.0	1992	1.454		General Expansion
				0.59	
West Owego (#1)	0.7	2000	0.545		General Expansion
				0.14	
Owego Village	1.0	1977	1.17		New Secondary Treatment Capability
		1977	0.67		Inflow Control
	3.0	1977	0.48		Micro-screening
				2.32	
Total Present Worth				15.56	

1. Eng. & Cont. @ 50%
2. 50 Years @ 6 1/8%

TABLE B-4

PLAN 2A

CAPITAL COSTS (MILLION $) AND OPERATION & MAINTENANCE COSTS ($\$10^3$/Yr)[1]
(Engineering & Contingencies @ 50%)

Year	Binghamton--Johnson City Capital	Binghamton--Johnson City O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	East Owego Capital	East Owego O&M	West Owego Capital	West Owego O&M	Owego Village Capital	Owego Village O&M
1977	8.95	830			1.81	310		140		40	2.32	70
1983					2.25	360						
1985												
1989												
1990												
1992							1.45	190				
1997	0.45											
2000									0.545	50		
2010												
2025	0.82											
2026						360		190		50		70
Present Worth (Million $)	9.13	12.9			3.38	5.1	0.59	2.4	0.14	0.6	2.32	1.1
Present Worth of Replacement (Million $)	6.3				3.8		0.8		0.2		0.5	
Total Present Worth (Million $)	28.3				12.3		3.8		0.94		3.9	

1. Eng. & Cont. @ 50%

TABLE B-5

PLAN 2A
PRESENT WORTH (MILLION $'s) COSTS[1]

	Bing-JC	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment						
Capital	1.7	1.6	0.6	0.1	1.1	5.1
O & M	7.8	4.4	1.9	0.4	0.7	15.2
Replacement	5.1	3.6	0.7	0.1	0.3	9.8
TOTAL	14.6	9.6	3.2	0.6	2.1	30.1
Interceptors						
Capital	2.9	1.8	-	-	-	4.7
O & M	-	-	-	-	-	-
Replacement	-	-	-	-	-	-
TOTAL	2.9	1.8	-	-	-	4.7
Storm Overflows						
Capital	4.2	-	-	-	1.2	5.4
O & M	0.8	-	-	-	0.1	0.9
Replacement	0.9	-	-	-	0.1	1.0
TOTAL	5.9	-	-	-	1.4	7.3
Infiltration Control						
Capital	0.2	-	-	-	-	0.2
O & M	-	-	-	-	-	-
Replacement	-	-	-	-	-	-
TOTAL	0.2	-	-	-	-	0.2
Sludge						
Capital	0.06	0.01	0.006	0.001	0.002	0.1
O & M	4.3	0.7	0.5	0.2	0.3	6.0
Replacement	0.3	0.2	0.1	0.1	0.1	0.8
TOTAL	4.7	0.9	0.6	0.3	0.4	6.9
Totals						
Capital	9.1	3.4	0.6	0.1	2.3	15.5
O & M	12.9	5.1	2.4	0.6	1.1	22.1
Replacement	6.3	3.8	0.8	0.2	0.5	11.6
TOTAL PLAN COST	28.3	12.3	3.8	0.9	3.9	49.2

[1] Based on 50 year economic life @ 6 1/8% and ENR = 2248.
Engineering and Contingencies @ 50%.

TABLE B-6

PLAN 2B CONSTRUCTION SCHEDULE[1]

Location	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[2] ($ Million)	Description
Binghamton-Johnson City	1.0	1977	0.24		Infiltration Control
	---	1977	4.13		Storm Overflow
	29.0	1977	0.06		Raw Wastewater Pumping
	22.2	1977	0.82		Aerator and Clarifier
	25.4	1991	0.82		Aerator and Clarifier
				5.61	
Endicott	0.8	1977	1.81		Nanticoke Valley Interceptor
	9.2	1983	2.25		Addl. Secondary Treatment Capacity
				3.38	
East Owego	3.0	1992	1.45		General Expansion
				0.59	
West Owego	0.7	2000	0.545		General Expansion
				0.14	
Owego Village	1.0	1977	1.17		New Secondary Treatment Capability
	3.0	1977	0.48		Micro-screening
		1977	0.67		Inflow Control
				2.32	
Chenango Valley	1.7	1977	2.19		New Secondary Treatment Plant
	2.2	1985	0.53		General Expansion
				2.52	
Total Present Worth				14.56	

1. Eng. & Cont. @ 50%
2. 50 years @ 6 1/8%

TABLE B-7

PLAN 2B

CAPITAL COSTS (MILLION $) AND OPERATION & MAINTENANCE COSTS ($\$10^3$/Yr)[1]
(Engineering & Contingencies @ 50%)

Year	Binghamton--Johnson City Capital	Binghamton--Johnson City O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	East Owego Capital	East Owego O&M	West Owego Capital	West Owego O&M	Owego Village Capital	Owego Village O&M
1977	5.25	770	2.19	130	1.81	310		140		40	2.32	70
1983					2.25	360						
1985			0.53	150								
1989												
1991	0.82											
1992							1.45	190				
1995												
2000									0.545	50		
2010												
2018												
2026		770		150		360		190		50		70
Present Worth (Million $)	5.61	11.9	2.52	2.1	3.38	5.1	0.59	2.4	0.14	0.6	2.32	1.1
Present Worth of Replacement (Million $)	6.1		0.5		3.8		0.8		0.2		0.5	
Total Present Worth (Million $)	23.6		5.1		12.3		3.8		0.94		3.9	

1. Eng. & Cont. @ 50%

TABLE B-8

PLAN 2B
PRESENT WORTH (MILLION $'s) COSTS[1]

	Chenango Valley	Bing-JC	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment							
Capital	2.0	1.2	1.6	0.6	0.1	1.1	6.6
O & M	1.5	7.3	4.4	1.9	0.4	0.7	16.2
Replacement	0.4	4.9	3.6	0.7	0.1	0.3	10.0
TOTAL	3.9	13.4	9.6	3.2	0.6	2.1	32.8
Interceptors							
Capital	-	-	1.8	-	-	-	1.8
O & M	-	-	-	-	-	-	-
Replacement	-	-	-	-	-	-	-
TOTAL	-	-	1.8	-	-	-	1.8
Storm Overflows							
Capital	-	4.2	-	-	-	1.2	5.4
O & M	-	0.8	-	-	-	0.1	0.9
Replacement	-	0.9	-	-	-	0.1	1.0
TOTAL	-	5.9	-	-	-	1.4	7.3
Infiltration Control							
Capital	-	0.2	-	-	-	-	0.2
O & M	-	-	-	-	-	-	-
Replacement	-	-	-	-	-	-	-
TOTAL	-	0.2	-	-	-	-	0.2
Sludge							
Capital	0.536	0.06	0.01	0.006	0.001	0.002	0.5
O & M	0.6	3.8	0.7	0.5	0.2	0.3	6.1
Replacement	0.1	0.3	0.2	0.1	0.1	0.1	0.9
TOTAL	1.2	4.1	0.9	0.6	0.3	0.4	7.5
Totals							
Capital	2.5	5.6	3.4	0.6	0.1	2.3	14.5
O & M	2.1	11.9	5.1	2.4	0.6	1.1	23.2
Replacement	0.5	6.1	3.8	0.8	0.2	0.5	11.9
TOTAL PLAN COST	5.1	23.6	12.3	3.8	0.9	3.9	49.6

[1] Based on 50 year economic life @ 6 1/8% and ENR = 2248.
Engineering and Contingencies @ 50%.

TABLE B-9

PLAN 2C CONSTRUCTION SCHEDULE[1]

Location	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[2] ($ Million)	Description
Binghamton-	1.0	1977	0.24		Infiltration Control
Johnson City	39.5	1977	4.13		Storm Overflow
	29.0	1977	0.06		Raw Wastewater Pumping
	22.2	1977	0.82		Aerator and Clarifier
	25.4	1991	0.82		Aerator and Clarifier
				5.61	
Endicott	0.8	1977	1.81		Nanticoke Valley Interceptor
	9.2	1983	2.25		Addl. Secondary Treatment Capacity
				3.38	
East Owego	3.0	1992	1.45		General Expansion
				0.59	
West Owego	0.7	2000	0.545		General Expansion
				0.14	
Owego Village	1.0	1977	1.17		New Secondary Treatment Capability
	3.0	1977	0.48		Micro-screening
		1977	0.67		Inflow Control
				2.32	
Chenango Valley	1.0	1977	1.5		New Secondary Treatment Plant
	2.2	1982	1.4		General Expansion
				2.5	
Total Present Worth				14.54	

1. Eng. & Cont. @ 50%
2. 50 years @ 6 1/8%

TABLE B-10

PLAN 2C

CAPITAL COSTS (MILLION $) AND OPERATION & MAINTENANCE COSTS ($\$10^3/Yr$)[1]
(Engineering & Contingencies @ 50%)

Year	Binghamton--Johnson City Capital	Binghamton--Johnson City O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	East Owego Capital	East Owego O&M	West Owego Capital	West Owego O&M	Owego Village Capital	Owego Village O&M
1977	5.25	770	1.5	86	1.81	310		140		40	2.32	70
1982			1.4	150								
1983					2.25	360						
1989												
1991	0.82											
1992							1.45	190				
1995									0.545	50		
2000												
2010												
2018												
2026		770		150		360		190		50		70
Present Worth (Million $)	5.61	11.9	2.5	2.1	3.38	5.1	0.59	2.4	0.14	0.6	2.32	1.1
Present Worth of Replacement (Million $)		6.1		0.5		3.8		0.8		0.2		0.5
Total Present Worth (Million $)		23.6		5.1		12.3		3.8		0.94		3.9

1. Eng. & Cont. @ 50%

TABLE B-11

PLAN 2C

PRESENT WORTH (MILLION $'s) COSTS[1]

	Chenango Valley	Bing-JC	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment							
Capital	2.0	1.2	1.6	0.6	0.1	1.1	6.6
O & M	1.5	7.3	4.4	1.9	0.4	0.7	16.2
Replacement	0.4	4.9	3.6	0.7	0.1	0.3	10.0
TOTAL	3.9	13.4	9.6	3.2	0.6	2.1	32.8
Interceptors							
Capital	---	---	1.8	---	---	---	1.8
O & M	---	---	---	---	---	---	---
Replacement	---	---	---	---	---	---	---
TOTAL	---	---	1.8	---	---	---	1.8
Storm Overflows							
Capital	---	4.2	---	---	---	1.2	5.4
O & M	---	0.8	---	---	---	0.1	0.9
Replacement	---	0.9	---	---	---	0.1	1.0
TOTAL	---	5.9	---	---	---	1.4	7.3
Infiltration Control							
Capital	---	0.2	---	---	---	---	0.2
O & M	---	---	---	---	---	---	---
Replacement	---	---	---	---	---	---	---
TOTAL	---	0.2	---	---	---	---	0.2
Sludge							
Capital	0.536	0.06	0.01	0.006	0.001	0.002	0.5
O & M	0.6	3.8	0.7	0.5	0.2	0.3	6.1
Replacement	0.1	0.3	0.2	0.1	0.1	0.1	0.9
TOTAL	1.2	4.1	0.9	0.6	0.3	0.4	7.5
Totals							
Capital	2.5	5.6	3.4	0.6	0.1	2.3	14.5
O & M	2.1	11.9	5.1	2.4	0.6	1.1	23.2
Replacement	0.5	6.1	3.8	0.8	0.2	0.5	11.9
TOTAL PLAN COST	5.1	23.6	12.3	3.8	0.9	3.9	49.6

[1] Based on 50 year economic life @ 6 1/8% and ENR = 2248.
Engineering and Contingencies @ 50%.

TABLE B-12

PLAN 3A CONSTRUCTION SCHEDULE[1]

Location	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[2] ($ Million)	Description
Binghamton-Johnson City	2.2	1977	2.88		Chenango Interceptor
	3.0	1977	1.39		Infiltration Control
	39.5	1977	4.13		Storm Overflow
	29.0	1977	0.06		Raw Wastewater Pumping
	26.9	1977	1.63		Aerator and Clarifier (2 sets)
	24.4	1994	4.67		Nitrification
				11.78	
Endicott	0.8	1977	1.81		Nanticoke Valley Interceptor
	9.2	1983	2.25		Addl. Secondary Treatment Capacity
				3.38	
East Owego	3.0	1992	1.45		General Expansion
				0.59	
West Owego	0.7	2000	0.545		General Expansion
				0.14	
Owego Village	1.0	1977	1.17		New Secondary Treatment Capability
	3.0	1977	0.48		Micro-screening
		1977	0.67		Inflow Control
				2.32	
Total Present Worth				18.21	

1. Eng. & Cont. @ 50%
2. 50 Years @ 6 1/8%

TABLE B-13

PLAN 3A

CAPITAL COSTS (MILLION $) AND OPERATION & MAINTENANCE COSTS ($\$10^3$/Yr)[1]
(Engineering & Contingencies @ 50%)

Year	Binghamton--Johnson City Capital	Binghamton--Johnson City O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	East Owego Capital	East Owego O&M	West Owego Capital	West Owego O&M	Owego Village Capital	Owego Village O&M
1977	10.09	830			1.81	310		140		40	2.32	70
1983					2.25	360						
1985												
1989												
1990												
1992							1.45	190				
1994	4.67	1000										
2000									0.545	50		
2010												
2018												
2026		1000				360		190		50		70
Present Worth (Million $)	11.78	13.7			3.38	5.1	0.59	2.4	0.14	0.6	2.32	1.1
Present Worth of Replacement (Million $)	6.5				3.8		0.8		0.2		0.5	
Total Present Worth (Million $)	32.0				12.3		3.8		0.94		3.9	

1. Eng. & Cont. @ 50%.

TABLE B-14

Plan 3A

PRESENT WORTH (MILLION $'s) COSTS[1]

	Bing-JC	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment						
Capital	3.2	1.6	0.6	0.1	1.1	6.6
O & M	8.6	4.4	1.9	0.4	0.7	16.0
Replacement	5.3	3.6	0.7	0.1	0.3	10.0
TOTAL	17.1	9.6	3.2	0.6	2.1	32.6
Interceptors						
Capital	2.9	1.8	---	---	---	4.7
O & M	---	---	---	---	---	---
Replacement	---	---	---	---	---	---
TOTAL	2.9	1.8	---	---	---	4.7
Storm Overflows						
Capital	4.2	---	---	---	1.2	5.4
O & M	0.8	---	---	---	0.1	0.9
Replacement	0.9	---	---	---	0.1	1.0
TOTAL	5.9	---	---	---	1.4	7.3
Infiltration Control						
Capital	1.4	---	---	---	---	1.4
O & M	---	---	---	---	---	---
Replacement	---	---	---	---	---	---
TOTAL	1.4	---	---	---	---	1.4
Sludge						
Capital	0.06	0.01	0.006	0.001	0.002	0.1
O & M	4.3	0.7	0.5	0.2	0.3	6.0
Replacement	0.3	0.2	0.1	0.1	0.1	0.8
TOTAL	4.7	0.9	0.6	0.3	0.4	6.9
Totals						
Capital	11.8	3.4	0.6	0.1	2.3	18.2
O & M	13.7	5.1	2.4	0.6	1.1	22.9
Replacement	6.5	3.8	0.8	0.2	0.5	11.8
TOTAL PLAN COST	32.0	12.3	3.8	0.9	3.9	52.9

[1] Based on 50 year economic life @ 6 1/8% and ENR = 2248.
Engineering and Contingencies @ 50%.

TABLE B-15

PLAN 3B CONSTRUCTION SCHEDULE[1]

Location	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[2] ($ Million)	Description
Binghamton-	3.0	1977	1.39		Infiltration Control
Johnson City	---	1977	4.13		Storm Overflow
	29.0	1977	0.06		Raw Wastewater Pumping
	21.5	1977	0.82		Aerator and Clarifier
	22.3	1994	4.37		Nitrification
	24.5	2000	0.82		Aerator and Clarifier
				8.2	
Endicott	0.8	1977	1.81		Nanticoke Valley Interceptor
	9.2	1983	2.25		Addl. Secondary Treatment Capacity
				3.38	
East Owego	3.0	1992	1.45		General Expansion
				0.59	
West Owego	0.7	2000	0.545		General Expansion
				0.14	
Owego Village	1.0	1977	1.17		New Secondary Treatment Capacity
	3.0	1977	0.48		Micro-screening
	---	1977	0.67		Inflow Control
				2.32	
Chenango Valley	1.7	1977	2.19		New Secondary Treatment Plant
	2.2	1985	0.53		General Expansion
				2.52	
Total Present Worth				17.1	

1. Eng. & Cont. @ 50%
2. 50 Years @ 6 1/8%

TABLE B-16

PLAN 3B

CAPITAL COSTS (MILLION $) AND OPERATION & MAINTENANCE COSTS ($\$10^3$/Yr)[1]
(Engineering & Contingencies @ 50%)

Year	Binghamton--Johnson City Capital	Binghamton--Johnson City O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	East Owego Capital	East Owego O&M	West Owego Capital	West Owego O&M	Owego Village Capital	Owego Village O&M
1977	6.40	770	2.19	130	1.81	310		140		40	2.32	70
1983					2.25	360						
1985			0.53	150								
1989												
1990												
1992							1.45	190				
1994	4.37	940										
2000	0.82								0.545	50		
2010												
2018												
2026		940		150		360		190		50		70
Present Worth (Million $)	8.2	12.8	2.5	2.1	3.4	5.1	0.6	2.4	0.1	0.6	2.3	1.1
Present Worth of Replacement (Million $)	6.4		0.5		3.8		0.8		0.2		0.5	
Total Present Worth (Million $)	27.4		5.1		12.3		3.8		0.9		3.9	

1. Eng. & Cont. @ 50%

TABLE B-17

Plan 3B

PRESENT WORTH (MILLION $'s) COSTS[1]

	Chenango Valley	Bing-JC	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment							
Capital	2.0	2.6	1.6	0.6	0.1	1.1	8.0
O & M	1.5	8.2	4.4	1.9	0.4	0.7	17.1
Replacement	0.4	5.2	3.6	0.7	0.1	0.3	10.3
TOTAL	3.9	16.0	9.6	3.2	0.6	2.1	35.4
Interceptors							
Capital	---	---	1.8	---	---	---	1.8
O & M	---	---	---	---	---	---	---
Replacement	---	---	---	---	---	---	---
TOTAL	---	---	1.8	---	---	---	1.8
Storm Overflows							
Capital	---	4.2	---	---	---	1.2	5.4
O & M	---	0.8	---	---	---	0.1	0.9
Replacement	---	0.9	---	---	---	0.1	1.0
TOTAL	---	5.9	---	---	---	1.4	7.3
Infiltration Control							
Capital	---	1.4	---	---	---	---	1.4
O & M	---	---	---	---	---	---	---
Replacement	---	---	---	---	---	---	---
TOTAL	---	1.4	---	---	---	---	1.4
Sludge							
Capital	0.536	0.06	0.01	0.006	0.001	0.002	0.6
O & M	0.6	3.8	0.7	0.5	0.2	0.3	6.1
Replacement	0.1	0.3	0.2	0.1	0.1	0.1	0.9
TOTAL	1.2	4.1	0.9	0.6	0.3	0.4	7.5
Totals							
Capital	2.5	8.2	3.4	0.6	0.1	2.3	17.1
O & M	2.1	12.8	5.1	2.4	0.6	1.1	24.1
Replacement	0.5	6.4	3.8	0.8	0.2	0.5	12.2
TOTAL PLAN COST	5.1	27.4	12.3	3.8	0.9	3.9	53.4

[1] Based on 50 year economic life @ 6 1/8% and ENR = 2248.
Engineering and Contingencies @ 50%.

TABLE B-18

PLAN 3C CONSTRUCTION SCHEDULE[1]

Location	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[2] ($ Million)	Description
Binghamton-Johnson City	3.0	1977	1.39		Infiltration Control
	39.5	1977	4.13		Storm Overflow
	29.0	1977	0.06		Raw Wastewater Pumping
	21.5	1977	0.82		Aerator and Clarifier
	22.3	1994	4.37		Nitrification
	24.5	2000	0.82		Aerator and Clarifier
				8.2	
Endicott	0.8	1977	1.81		Nanticoke Valley Interceptor
	9.2	1983	2.25		Addl. Secondary Treatment Capacity
				3.38	
East Owego	3.0	1992	1.45		General Expansion
				0.59	
West Owego	0.7	2000	0.545		General Expansion
				0.14	
Owego Village	1.0	1977	1.17		New Secondary Treatment Capacity
	3.0	1977	0.48		Micro-screening
	---	1977	0.67		Inflow Control
				2.32	
Chenango Valley	1.0	1977	1.5		New Secondary Treatment Plant
	2.2	1985	1.4		General Expansion
				2.37	
Total Present Worth				17.00	

1. Eng. & Cont. @ 50%

2. 50 years @ 6 1/8%

TABLE B-19

PLAN 3C

CAPITAL COSTS (MILLION \$) AND OPERATION & MAINTENANCE COSTS (\$$10^3$/Yr)[1]
(Engineering & Contingencies @ 50%)

Year	Binghamton--Johnson City Capital	Binghamton--Johnson City O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	East Owego Capital	East Owego O&M	West Owego Capital	West Owego O&M	Owego Village Capital	Owego Village O&M
1977	6.40	770	1.5	86	1.81	310		140		40	2.32	70
1982			1.4	150								
1983					2.25	360						
1989												
1990												
1992							1.45	190				
1994	4.37	940										
2000	0.82								0.545	50		
2010												
2018												
2026		940		150		360		190		50		70
Present Worth (Million \$)	8.2	12.8	2.5	2.1	3.4	5.1	0.6	2.4	0.1	0.6	2.3	1.1
Present Worth of Replacement (Million \$)	6.4		0.5		3.8		0.8		0.2		0.5	
Total Present Worth (Million \$)	27.4		5.1		12.3		3.8		0.9		3.9	

1. Eng. & Cont. @ 50%

TABLE B-20

PLAN 3C

PRESENT WORTH (MILLION $'s) COSTS[1]

	Chenango Valley	Bing-JC	Endicott	East Owego	West Owego	Owego Village	TOTAL
Wastewater Treatment							
Capital	2.0	2.6	1.6	0.6	0.1	1.1	8.0
O & M	1.5	8.2	4.4	1.9	0.4	0.7	17.1
Replacement	0.4	5.2	3.6	0.7	0.1	0.3	10.3
TOTAL	3.9	16.0	9.6	3.2	0.6	2.1	35.4
Interceptors							
Capital	---	---	1.8	---	---	---	1.8
O & M	---	---	---	---	---	---	---
Replacement	---	---	---	---	---	---	---
TOTAL	---	---	1.8	---	---	---	1.8
Storm Overflows							
Capital	---	4.2	---	---	---	1.2	5.4
O & M	---	0.8	---	---	---	0.1	0.9
Replacement	---	0.9	---	---	---	0.1	1.0
TOTAL	---	5.9	---	---	---	1.4	7.3
Infiltration Control							
Capital	---	1.4	---	---	---	---	1.4
O & M	---	---	---	---	---	---	---
Replacement	---	---	---	---	---	---	---
TOTAL	---	1.4	---	---	---	---	1.4
Sludge							
Capital	0.536	0.06	0.01	0.006	0.001	0.002	0.5
O & M	0.6	3.8	0.7	0.5	0.2	0.3	6.1
Replacement	0.1	0.3	0.2	0.1	0.1	0.1	0.9
TOTAL	1.2	4.1	0.9	0.6	0.3	0.4	7.5
Totals							
Capital	2.5	8.2	3.4	0.6	0.1	2.3	17.1
O & M	2.1	12.8	5.1	2.4	0.6	1.1	24.1
Replacement	0.5	6.4	3.8	0.8	0.2	0.5	12.2
TOTAL PLAN COST	5.1	27.4	12.3	3.8	0.9	3.9	53.4

1 Based on 50 year economic life @ 6 1/8% and ENR = 2248. Engineering and Contingencies @ 50%.

TABLE B-21

PLAN 4 CONSTRUCTION SCHEDULE

Location	New Total Capacity (MGD)	Year	Cost ($ Million)	Present Worth[2] ($ Million)	Description
Binghamton-	2.2	1977	2.88		Chenango Interceptor
Johnson City	3.0	1977	1.39		Infiltration Control
	39.5	1977	4.13		Storm Overflow
	24.4	1977	1.64		Aeration and Clarification
	29.0	1977	0.06		Raw Wastewater Pumping
	20.5	1983	4.26		Nitrification
	20,5	1985	9.22		AWT Add-Ons
				18.81	
Endicott	0.8	1977	1.81		Nanticoke Valley Interceptor
	7.5	1983	2.60		Nitrification
	7.5	1985	4.13		AWT Add-Ons
				6.2	
East Owego	2.0	1983	0.97		Nitrification
	2.5	1985	1.82		AWT Add-Ons
	2.5	2000	1.21		Secondary & Nitrification
				2.12	
West Owego	0.9	1977	0.76		Pressure Main from Owego Village
		1977	0.67		Inflow Control
	3.0	1977	0.48		Micro-screening
	1.5	1977	1.49		General Expansion
	1.5	1983	0.78		Nitrification
	1.5	1985	1.38		AWT Add-Ons
				4.81	
Total Present Worth				31.9	

1. Eng. & Cont. A 50%
2. 50 Years @ 6 1/8%

TABLE B-22

PLAN 4

CAPITAL COSTS (MILLION $) AND OPERATION & MAINTENANCE COSTS ($\$10^3$/Yr)[1]
(Engineering & Contingencies @ 50%)

Year	Binghamton--Johnson City Capital	Binghamton--Johnson City O&M	Chenango Capital	Chenango O&M	Endicott Capital	Endicott O&M	East Owego Capital	East Owego O&M	West Owego Capital	West Owego O&M	Owego Village Capital	Owego Village O&M
1977	10.10	830			1.81	310		140	3.40	92		
1983	4.26	1000			2.60	400	0.97	180	0.78	132		
1985	9.22	2470			4.13	1010	1.82	440	1.38	312		
1989												
1990												
1992												
1995												
2000							1.21					
2010												
2018												
2027		2470				1010		650		312		
Present Worth (Million $)	18.8	28.3			6.2	11.4	2.1	5.0	4.8	3.7		
Present Worth of Replacement (Million $)	8.0				4.4		1.1		.8			
Total Present Worth (Million $)	55.1				22.0		8.2		9.3			

1 Engineering and Contingencies @ 50%

TABLE B-23

PLAN 4
PRESENT WORTH (MILLION $'s) COSTS[1]

	Bing-JC	Endicott	East Owego	West Owego	TOTAL
Wastewater Treatment					
Capital	10.2	4.4	2.1	2.8	19.5
O & M	21.5	10.0	4.3	3.1	38.9
Replacement	6.8	4.2	1.0	0.6	12.6
TOTAL	38.5	18.6	7.4	6.5	71.0
Interceptors					
Capital	2.9	1.8	---	0.8	5.5
O & M	---	---	---	---	---
Replacement	---	---	---	---	---
TOTAL	2.9	1.8	---	0.8	5.5
Storm Overflows					
Capital	4.2	---	---	1.2	5.4
O & M	0.8	---	---	0.1	0.9
Replacement	0.9	---	---	0.1	1.0
TOTAL	5.9	---	---	1.4	7.3
Infiltration Control					
Capital	1.4	---	---	---	1.4
O & M	---	---	---	---	---
Replacement	---	---	---	---	---
TOTAL	1.4	---	---	---	1.4
Sludge					
Capital	0.09	0.02	0.01	0.01	0.1
O & M	6.0	1.4	0.7	0.5	8.6
Replacement	0.3	0.2	0.1	0.1	0.7
TOTAL	6.4	1.6	0.8	0.6	9.4
Totals					
Capital	18.8	6.2	2.1	4.8	31.9
O & M	28.3	11.4	5.0	3.7	48.4
Replacement	8.0	4.4	1.1	0.8	14.3
TOTAL PLAN COST	55.1	22.0	8.2	9.3	94.6

[1] Based on 50 year economic life @ 6 1/8% and ENR = 2248. Engineering and Contingencies @ 50%.

TABLE B-24

FINAL PLANS COST SUMMARY [1]

(Million $'s)

Cost*	1	2A	2B	2C	3A	3B	3C	4
PRESENT WORTH								
Capital	3.0	15.5	14.5	14.5	18.2	17.1	17.1	31.9
O&M	19.5	22.1	23.2	23.2	22.9	24.1	24.1	48.4
Replacement	9.8	11.6	11.9	11.9	11.8	12.2	12.2	14.3
TOTAL PRESENT WORTH	32.3	49.2	49.6	49.6	52.9	53.4	53.4	94.6
AVERAGE ANNUAL COST	2.1	3.2	3.2	3.2	3.4	3.5	3.5	6.1

*6 1/8 percent interest and 50-year project life.

1. Eng. & Cont. @ 50%

TABLE B-25

TOTAL COST COMPARISON*

30 Percent and 50 Percent Engineering and Contingencies Factors
($ Millions)

	PLAN							
	1	2A	2B	2C	3A	3B	3C	4
30% Engineering & Contingencies								
Capital	2.6	13.5	12.6	12.6	15.8	14.8	14.8	27.5
O&M	19.5	22.1	23.2	23.2	22.9	24.1	24.1	48.4
Replacement	9.8	11.6	11.9	11.9	11.8	12.2	12.2	14.3
Total	31.9	47.2	47.7	47.7	50.5	51.1	51.1	90.2
50% Engineering & Contingencies								
Capital	3.0	15.5	14.5	14.5	18.2	17.1	17.1	31.9
O&M	19.5	22.1	23.2	23.2	22.9	24.1	24.1	48.4
Replacement	9.8	11.6	11.9	11.9	11.8	12.2	12.2	14.3
Total	32.3	49.2	49.6	49.6	52.9	53.4	53.4	94.6

*Present Worth, 6 1/8 percent interest, 50-year economic life.

ADDENDUM C

FLOOD PROOFING FOR SEWAGE TREATMENT PLANTS

PURPOSE

A major aim of the Flood Disaster Protection Act of 1973 is that new construction undertaken within an area having flood hazards should be properly elevated or flood-proofed to reduce or avoid damage. States and local communities are required, as a condition of future Federal financial assistance, to participate in the flood insurance program and to adopt adequate flood plain ordinances and enforcement provisions consistent with Federal standards to reduce or avoid future flood losses. As the plans presented in Chapter VIII did not include flood proofing measures, the purpose of this addendum is to present a preliminary analysis of the design and construction costs of flood proofing structures for each of the final plans.

METHODOLOGY

The Federal Insurance Administration has adopted the 100-year flood as the standard for the identification of special flood hazard areas and as the flood elevation for the adoption of local land use controls. The 100-year standard represents the flood level that on the average will have a 1 percent chance of being equalled or exceeded in any given year.

The 100-year flood elevation at each sewage treatment plant (STP) was obtained from Flood Plain Information Reports prepared by the US Army Corps of Engineers in 1969 (Broome County) and 1975 (Tioga County). The flood-proofing structures consisted of either an earth levee or a concrete wall around the STP's. The height of the flood structure was determined from the difference between the 100-year flood elevation and the average ground elevation of each STP (from USGS topographic maps), plus 3 feet of freeboard (Table C-1). Because of their lower cost, earth levees were used except where insufficient space was available, in which event a concrete wall was selected. All flood walls and earth levees were based on typical Corps designs for flood proofing structures.

Because this was a preliminary analysis only, on-site survey investigations were not conducted. The final wastewater management plans were studied to determine the necessary length of flood structure for each plan. The levees or concrete walls were located immediately outside the STP facilities including the proposed STP additions as outlined in each wastewater management plan. A closure structure was assumed for roads loading into the STP's where the height of the structure exceeded 10 feet or was of concrete design.

Some problems may be encountered in flood wall construction at the Binghamton-Johnson City, Owego No. 2, and Owego Village plants because of the close proximity of these STP's to rivers and roads. At the Town of Owego STP No. 2 and at the Chenango Valley site, the existing embankments for Routes 17 and 81, respectively, may reduce the necessary length of the levees. However, the possible protection afforded by the highway embankments was not used in the analysis and the flood structures were designed around the entire perimeter of the STP's.

Flood-proofing designs for each STP under the conditions of each plan are presented in the following sections.

BINGHAMTON-JOHNSON CITY STP

To protect the Binghamton-Johnson City STP from the 100-year flood, a wall 6 feet above the original ground surface would be required. Because of the severe space restrictions at the site under any of the plans, a concrete wall was selected rather than an earth levee. A typical "Ell" section concrete wall is shown in Figure C-1. The approximate location of the 1-foot thick flood wall for each plan is shown in Figures C-2 through C-5.

Required quantities of construction materials were based on typical Corps of Engineers design criteria. The design values, expressed on a per linear foot of concrete wall basis, together with the length of the wall, were used in estimating the total quantity of materials needed for each plan as shown in Table C-2. These estimated quantities were used to prepare the bid schedule shown in Table C-3. Unit Costs were also based on typical design values.

ENDICOTT STP

To protect the Endicott STP from the 100-year flood, a structure 22.5 feet above the original ground surface would be required. An earth levee was selected as it was found to be about one third the cost of a concrete wall. A cross-section of the levee based on typical Corps' design criteria is shown in Figure C-6. For additional protection, riprap would be placed on the sides of the levee nearest Nanticoke Creek and the Susquehanna River.

The approximate location of the 120-foot wide levee for each plan is shown in Figures C-7 and C-8.

Required quantities of construction materials were based on the levee design shown in Figure C-6. The design values, expressed on a per linear foot of wall basis, together with the length of the wall were used in estimating the total quantity of materials needed for each plan as shown in Table C-4. These estimated quantities were used to prepare the bid schedule shown in Table C-5. Unit Costs were also based on typical design values and are not specific to the study area.

CHENANGO VALLEY STP

To protect the Chenango Valley STP from the 100-year flood, a structure 8.5 feet above the original ground surface would be required. Because of its comparatively low cost, an earth levee was selected rather than a concrete wall. A cross-section of the levee, based on typical Corps' design criteria, is shown in Figure C-9.

The approximate location of the 52-foot wide levee for each plan is shown in Figure C-10.

The existing embankment for Interstate 81 may provide a degree of protection by reducing the velocity of flood flows. However, the design and cost of the levee assumed a wall around the entire STP perimeter and did not take into account any protection from the highway embankment.

Required quantities of construction materials were based on the levee design shown in Figure C-9. The design values,

expressed on a per linear foot of wall basis, together with the length of the wall were used in estimating the total quantity of materials needed for each plan as shown in Table C-6. These estimated quantities were used to prepare the bid schedule shown in Table C-7. Unit costs were also based on typical design values and are not specific to the study area.

TOWN OF OWEGO STP No. 1

To protect the Town of Owego STP No. 1 from the 100-year flood, a structure 6 feet above the original ground surface would be required. Because of its comparatively low cost, an earth levee was selected rather than a concrete wall. A cross-section of the levee, based on typical Corps' design criteria, is shown in Figure C-11. The approximate location of the 39-foot wide levee for each plan is shown in Figures C-12 and C-13.

Required quantities of construction materials were based on the levee design shown in Figure C-11. The design values, expressed on a per linear foot of wall basis, together with the length of the wall were used in estimating the total quantity of materials neded for each plan as shown in Table C-8. These estimated quantities were used to prepare the bid schedule shown in Table C-9. Unit costs were also based on typical design values and are not specific to the study area.

TOWN OF OWEGO STP No. 2

To protect the Town of Owego STP No. 2 from the 100-year flood, a structure 12.5 feet above the original ground surface would be required. Because of its comparatively low cost, an earth levee was selected rather than a concrete wall. However, due to the additional facilities in Plan 4, a portion of the wall around the STP would have to be concrete in order to allow room for the existing roadway.

A cross-section of the levee, based on typical Corps' design criteria, is shown in Figure C-14. A cross-section of the

concrete Ell wall to be used as part of the flood protection works under Plan 4 is shown in Figure C-15. The approximate location of the flood wall for each plan is shown in Figures C-16 and C-17.

The design and cost of the levee assumed a wall around the entire STP perimeter. The existing embankment for Route 17 may provide a degree of protection by reducing the velocity of flood flows. However, the design and cost of the levee assumed a wall around the entire STP perimeter and did not take into account any protection from the road embankment.

Required quantities of construction materials were based on the designs shown in Figures C-14 and C-15. The design values, expressed on a per linear foot of wall basis, together with the length of the wall, were used in estimating the total quantity of materials needed for each plan as shown in Table C-10. These estimated quantities were used to prepare the bid schedule shown in Table C-11. Unit costs were also based on typical design values and are not specific to the study area.

OWEGO VILLAGE STP

To protect the Owego Village STP from the 100-year flood, a structure 14.5 feet above the original ground surface would be required. Because of its comparatively low cost, an earth levee was selected rather than a concrete wall. A cross-section of the levee, based on typical Corps' design criteria, is shown in Figure C-18. The approximate location of the flood wall for each plan is shown in Figure C-19.

Required quantities of construction materials were based on the design shown in Figure C-18. The design values, expressed on a per linear foot of wall basis, together with the length of the wall, were used in estimating the total quantity of materials needed for each plan as shown in Table C-12. These estimated quantities were used to prepare the bid schedule shown in Table C-13. Unit costs were also based on typical design values and are not specific to the study area.

SUMMARY OF COSTS

A summary of the estimated costs for flood-proofing sewage treatment plants under the conditions of each wastewater management plan is presented below. Table C-14 shows the cost of flood proofing on a per linear foot of wall basis and Table C-15 gives the length of the levee or concrete wall. Table C-16 presents the total costs of flood proofing each STP for each plan.

As indicated in the tables, the cost of providing flood protection to an STP does not vary significantly between wastewater management plans. The costs presented in Table C-16 includes only flood-proofing measures and do not include any wastewater management items.

Table C-1

100-YEAR FLOOD ELEVATIONS AND HEIGHT OF FLOOD WALLS

STP	100-Year Flood Elevation[1] Feet Above MSL	Average STP Elevation[2] Ft Above MSL	Design Height of Flood Wall Feet[3]	Type of Structure
Broome County				
Binghamton-Johnson City	838.0	835.0	6.0	Concrete Wall
Endicott	829.5	810.0	22.5	Earth Levee
Chenango Valley	850.5	845.0	8.5	Earth Levee
Tioga County				
Owego Town No. 1	818.0	815.0	6.0	Earth Levee
Owego Town No. 2	824.5	815.0	12.5	Earth Levee & Concrete Wall
Owego Village	816.5	805.0	14.5	Earth Levee

1. Flood Flood Plain Information Reports for Broome County (1969) and Tioga County (1975), US Army Corps of Engineers, Baltimore District
2. From USGS Topographic Maps
3. Includes 3 feet of freeboard. Does not include depth beneath original ground surface.

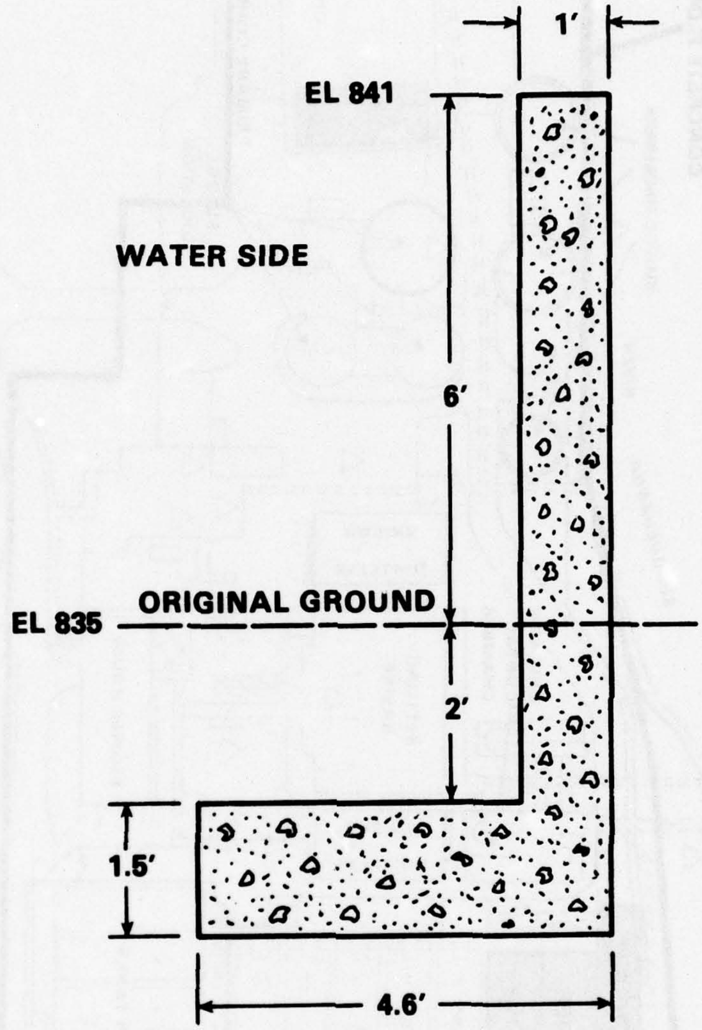


FIGURE C-1. BINGHAMTON-JOHNSON CITY STP CONCRETE ELL WALL SECTION

BINGHAMTON-JOHNSON CITY STP
PLAN 2A FLOOD STRUCTURE

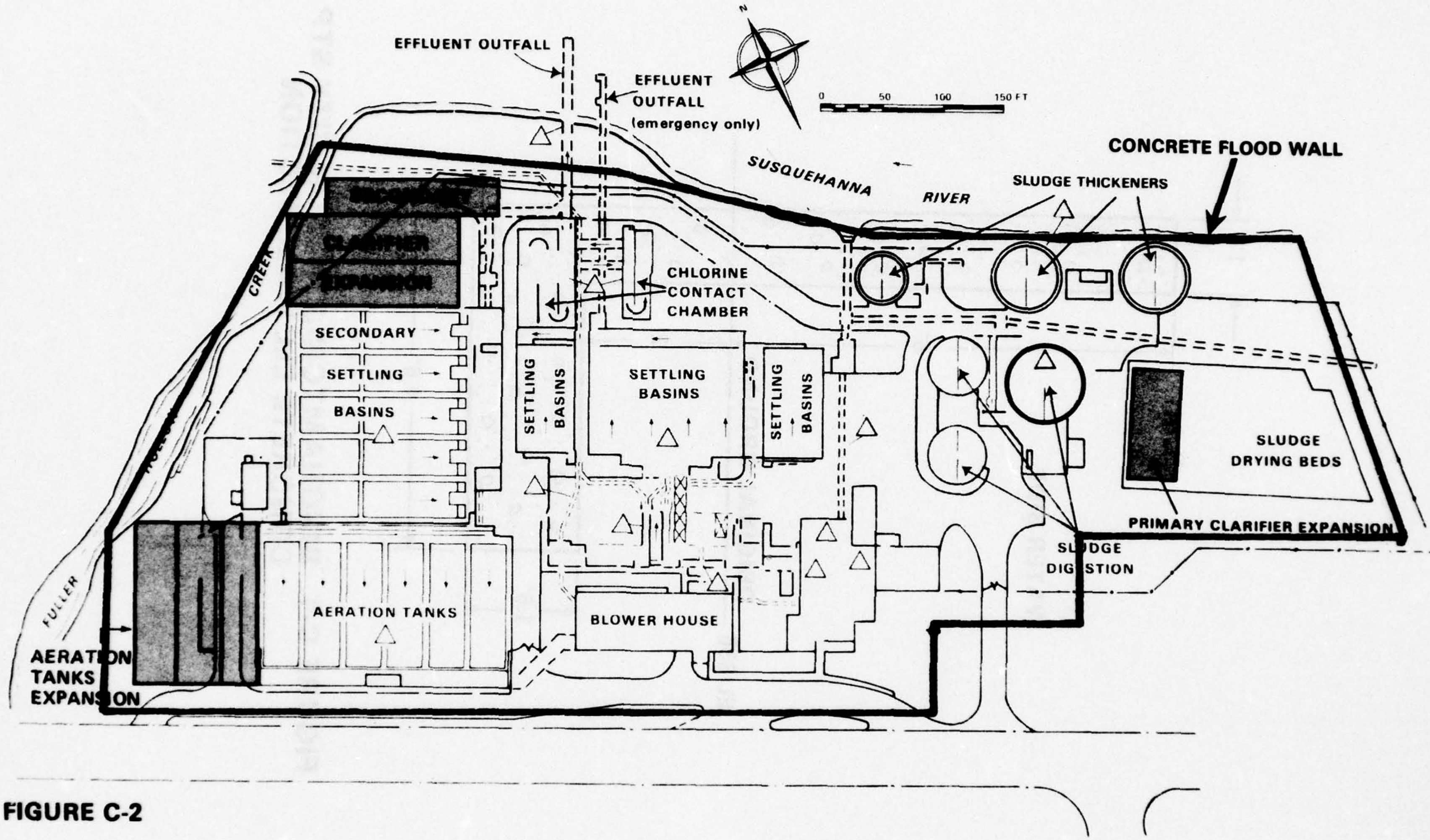


FIGURE C-2

BINGHAMTON-JOHNSON CITY STP
PLANS 2B, 2C FLOOD STRUCTURE

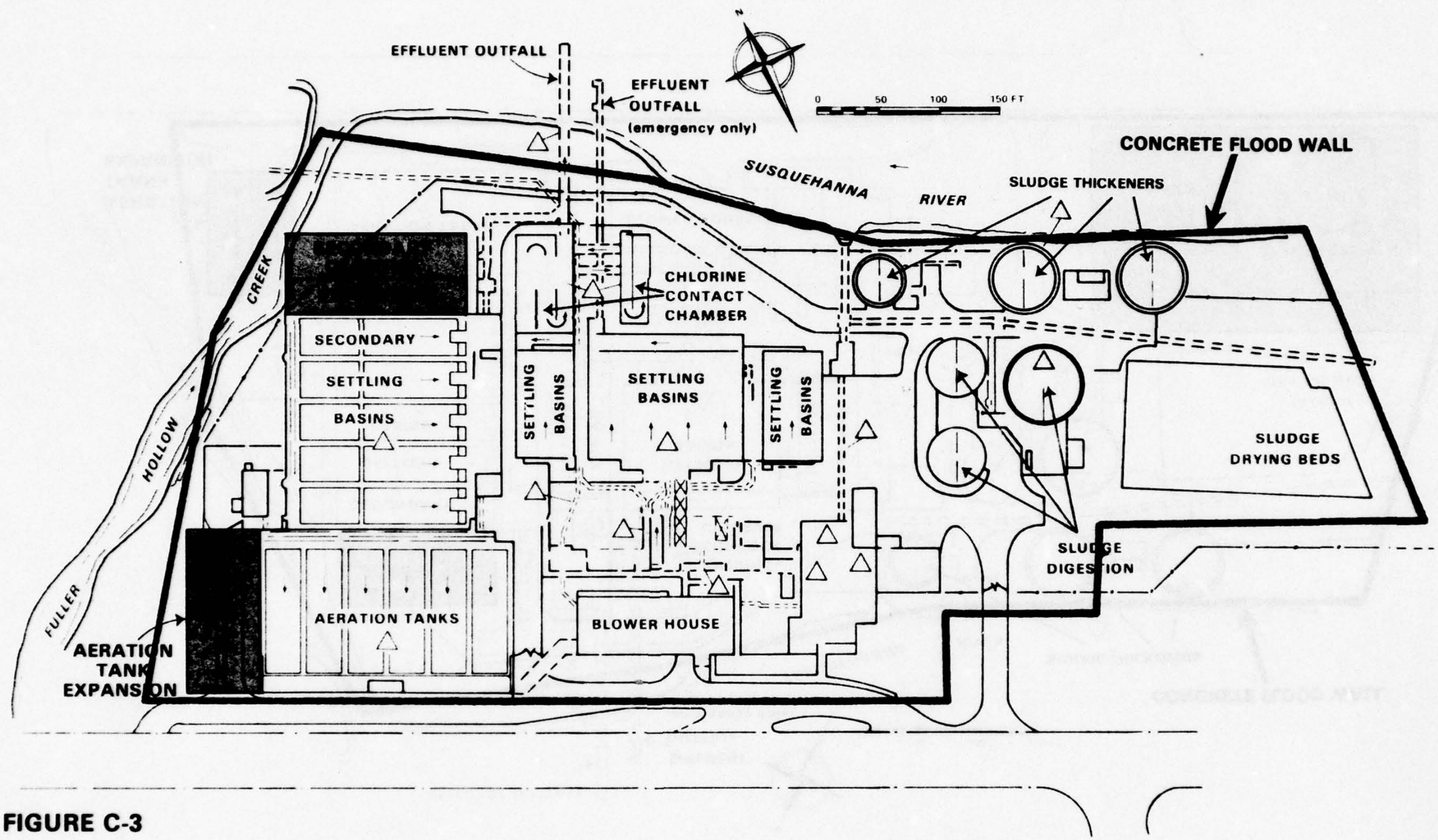


FIGURE C-3

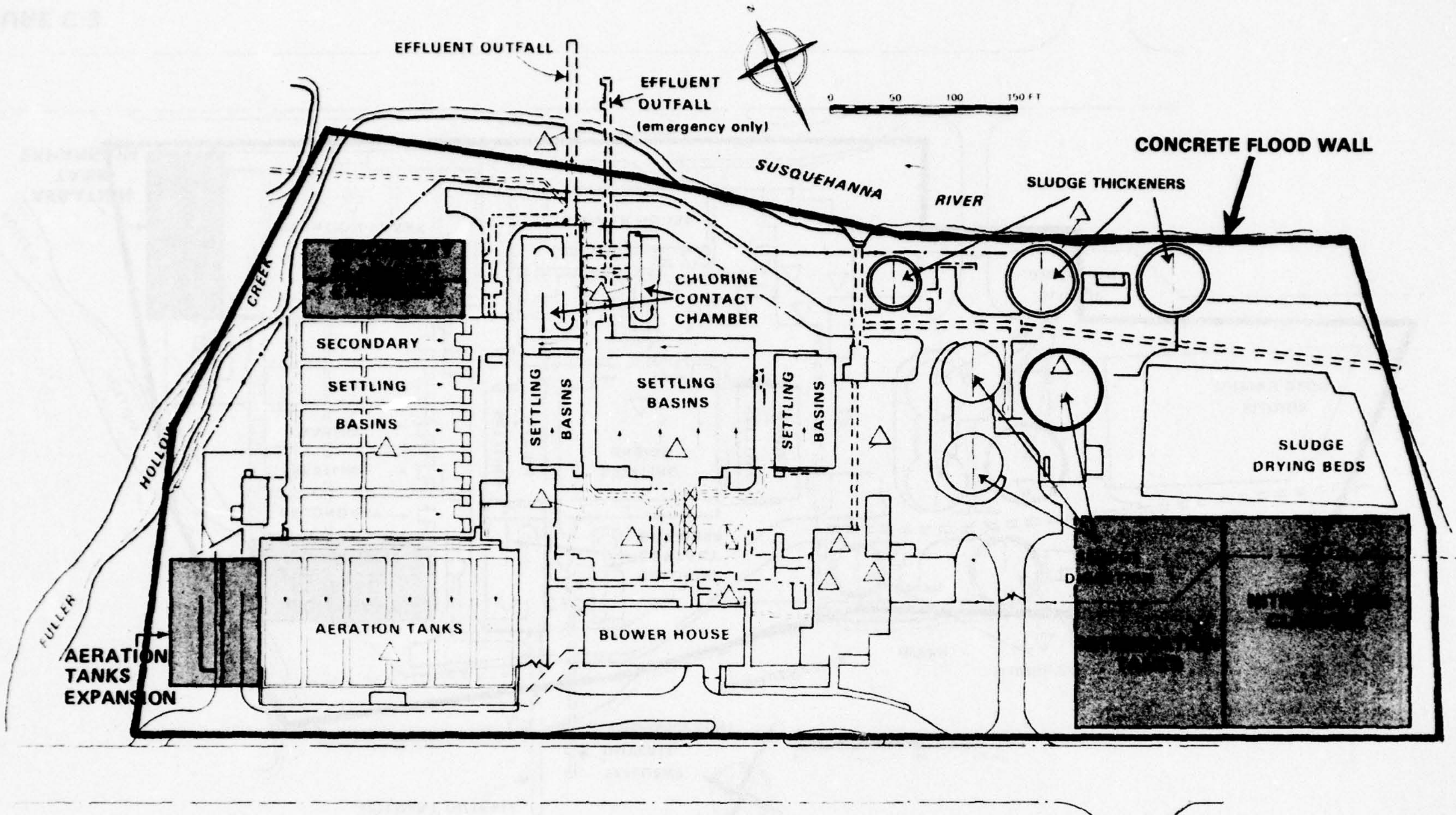


FIGURE C-4

BINGHAMTON-JOHNSON CITY STP
PLANS 3A, 3B, 3C FLOOD STRUCTURE

BINGHAMTON-JOHNSON CITY STP, PLAN 4 FLOOD STRUCTURE

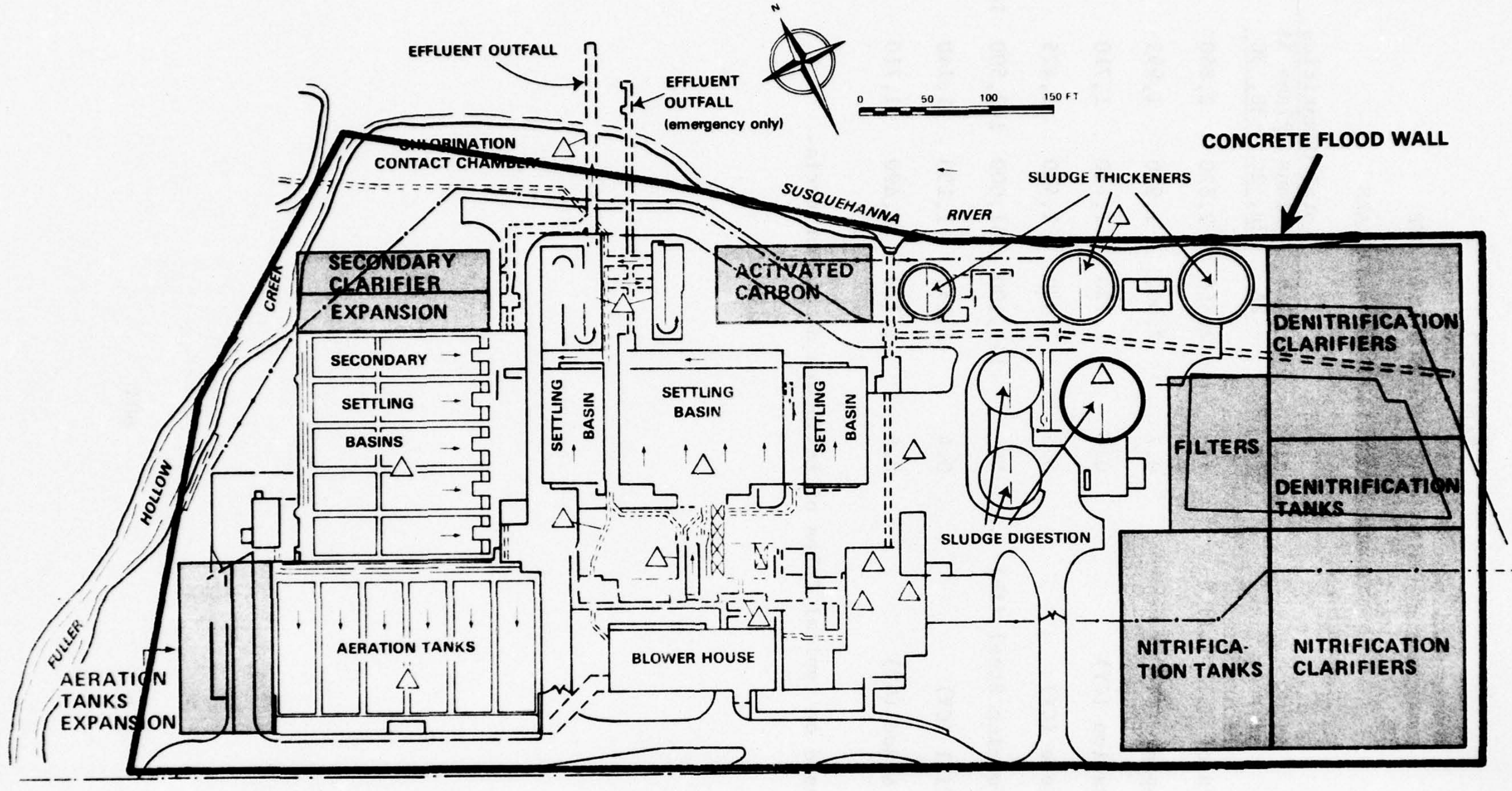


FIGURE C-5

TABLE C-2

BINGHAMTON-JOHNSON CITY STP

FLOODWALL CONSTRUCTION MATERIALS

Item	Unit Design Value[1] (Per LF)	Total Quantities Plan 2A	Plans 2B, 2C	Plans 3A 3B, 3C	Plan 4
Length of Wall (LF)	---	2,860	2,820	2,850	2,925
Stripping (SY)	0.7	2,000	1,970	1,995	2,050
Excavation (CY)	0.6	1,720	1,690	1,710	1,755
Concrete (CY)	0.5	1,430	1,410	1,425	1,460
Reinforcing Steel (lb)	50	143,100	141,000	142,500	146,200
Backfill (CY)	0.4	1,145	1,130	1,140	1,170
Seed & Sod (SY)	0.6	1,720	1,690	1,710	1,755

1. Based on typical Corps of Engineers design criteria.

TABLE C-3

BINGHAMTON-JOHNSON CITY STP -- FLOODWALL CONSTRUCTION BID SCHEDULE

Item	Unit Price $	Unit	PLAN 2A Quantity	PLAN 2A Cost $	PLANS 2B, 2C Quantity	PLANS 2B, 2C Cost $	PLANS 3A, 3B, 3C Quantity	PLANS 3A, 3B, 3C Cost $	PLAN 4 Quantity	PLAN 4 Cost $
1. Stripping	0.25	SY	2,000	500	1,970	500	1,995	500	2,050	500
2. Excavation	10.00	CY	1,720	17,200	1,690	17,000	1,710	17,100	1,755	17,600
3. Concrete	190.00	CY	1,430	271,900	1,410	267,900	1,425	270,800	1,460	277,800
4. Reinforcing Steel	0.45	lb.	143,100	64,400	141,000	63,500	142,500	64,100	146,250	65,800
5. Backfill	8.00	CY	1,145	9,200	1,130	9,000	1,140	9,100	1,170	9,400
6. Seeding & Sodding	0.31	SY	1,720	500	1,690	500	1,710	500	1,755	500
7. Closure Structure	28,000	LS	1	28,000	1	28,000	1	28,000	1	28,000
Subtotal				$392,000		$386,000		$390,000		$399,000
Engineering & Contingencies @ 30%				117,000		116,000		117,000		120,000
TOTAL				$509,000		$502,000		$507,000		$519,000

All costs in April 1975 dollars.

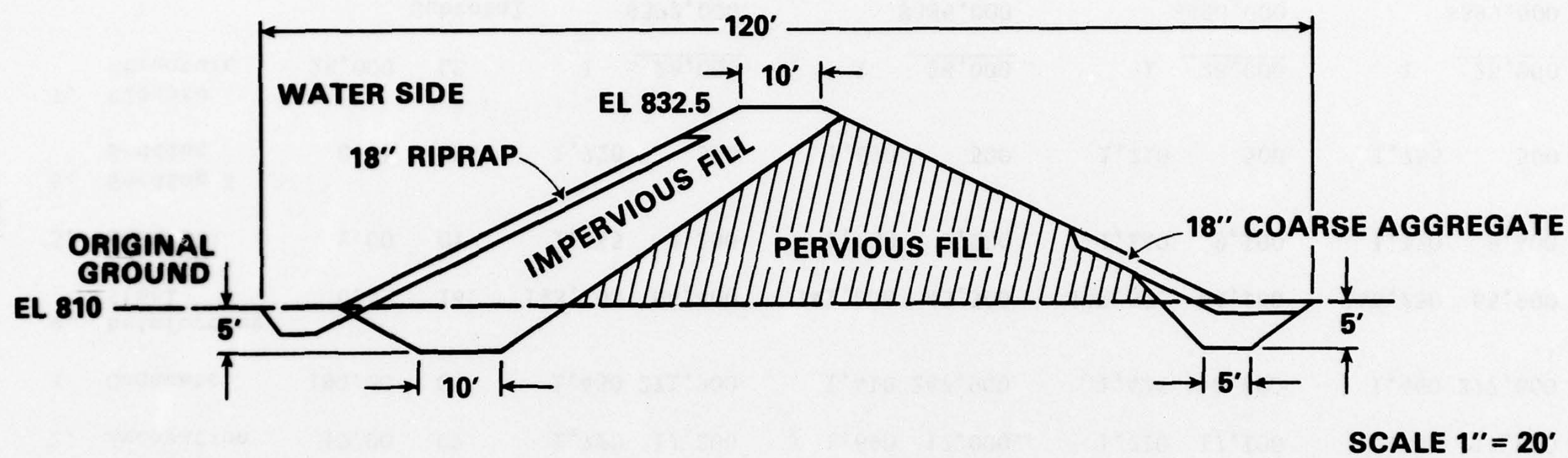


FIGURE C-6. ENDICOTT STP EARTH LEVEE CROSS SECTION

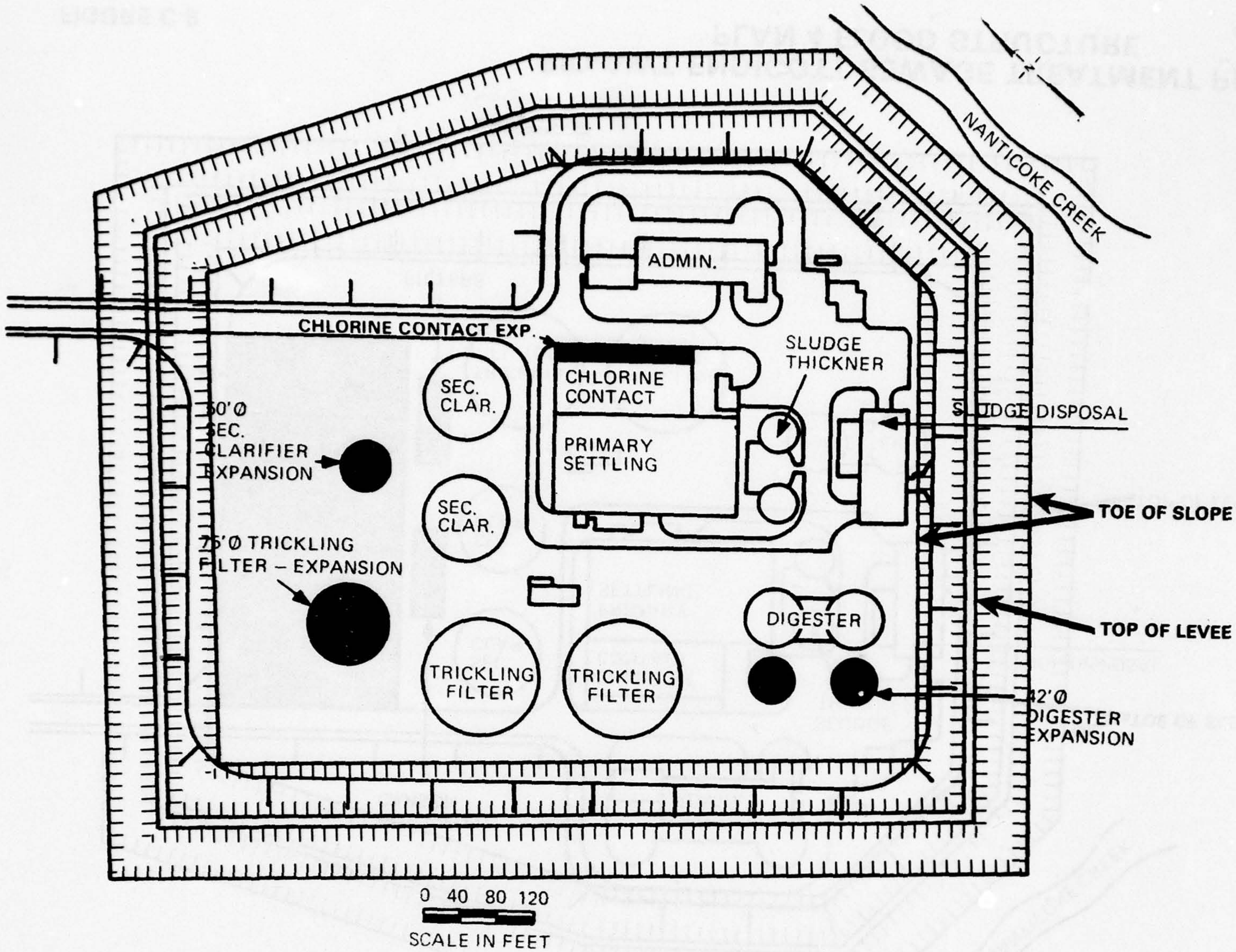


FIGURE C-7

ENDICOTT SEWAGE TREATMENT PLANT
PLANS 2A, 2B, 2C, 3A, 3B, 3C FLOOD STRUCTURE

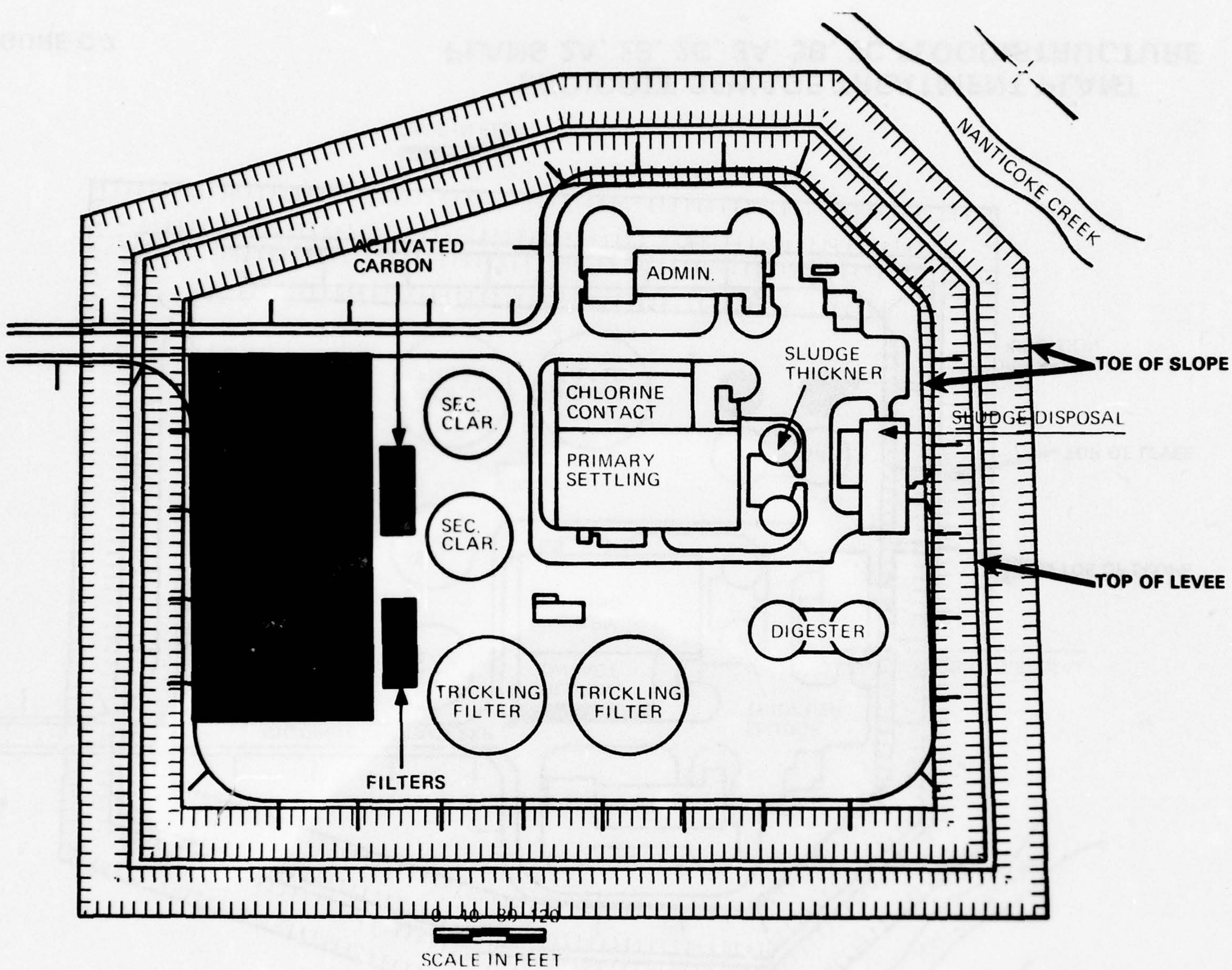


FIGURE C-8

BIO-AWT ENDICOTT SEWAGE TREATMENT PLANT
PLAN 4 FLOOD STRUCTURE

TABLE C-4

ENDICOTT STP

FLOOD LEVEE CONSTRUCTION MATERIALS

	Unit Design Value[1] (Per LF)	Total Quantities: Plans 2A, 2B, 2C 3A, 3B, 3C	Total Quantities: Plan 4
Length of Levee (LF)	---	3,100	3,240
Stripping (SY)	13.3	41,230	43,090
Excavation (CY)	5.3	16,430	17,170
Impervious Fill (CY)	19.8	61,380	64,150
Pervious Fill (CY)	28	86,800	90,720
Coarse Aggregate (CY)	1.6	4,960	5,180
Riprap (CY)	3.2	7,870[2]	8,260[3]
Drainage Fill (CY)	1.7	5,270	5,510
Seeding & Sodding (SY)			
With Riprap	5.3	13,040[2]	13,670[3]
Without Riprap	10.1	6,460	6,670

1. Based on typical Corps of Engineers design criteria.

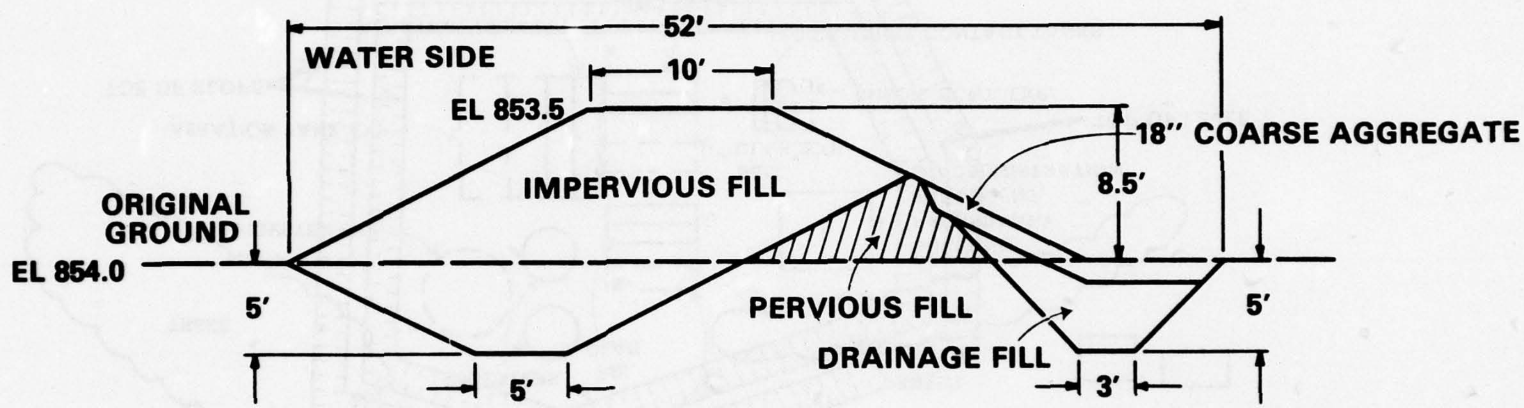
2. Length of levee to have riprap is 2,460 feet.

3. Length of levee to have riprap is 2,580 feet.

TABLE C-5
ENDICOTT STP
FLOOD LEVEE CONSTRUCTION BID SCHEDULE

ITEM	UNIT PRICE	UNIT	PLAN 2A, 2B, 2C, 3A, 3B, 3C		PLAN 4	
			QUANTITY	COST $	QUANTITY	COST $
1. Stripping	0.25	SY	41,230	10,300	43,090	10,800
2. Excavation	6.00	CY	16,430	98,600	17,170	103,000
3. Impervious Fill	7.00	CY	61,380	429,700	64,150	449,100
4. Pervious Fill	2.50	CY	86,800	217,000	90,720	226,800
5. Coarse Aggregate	18.00	CY	4,960	89,300	5,180	93,300
6. Riprap - 18"	30.00	CY	7,870	236,200	8,260	247,700
7. Drainage Fill	13.00	CY	5,270	68,500	5,510	71,600
8. Seeding & Sodding	0.31	SY	19,500	6,000	20,340	6,300
9. Closure Structure (Ht. - 10')	65,000	EA	1	65,000	1	65,000
	SUBTOTAL			$1,221,000		$1,274,000
	ENGINEERING & CONTINGENCIES @ 30%			366,000		382,000
	TOTAL			$1,587,000		$1,656,000

All costs on April 1975 dollars



SCALE 1″= 10′

FIGURE C-9. CHENANGO VALLEY STP EARTH LEVEE CROSS SECTION

CHENANGO VALLEY STP
PLANS 2B, 2C, 3B and 3C
FLOOD STRUCTURE

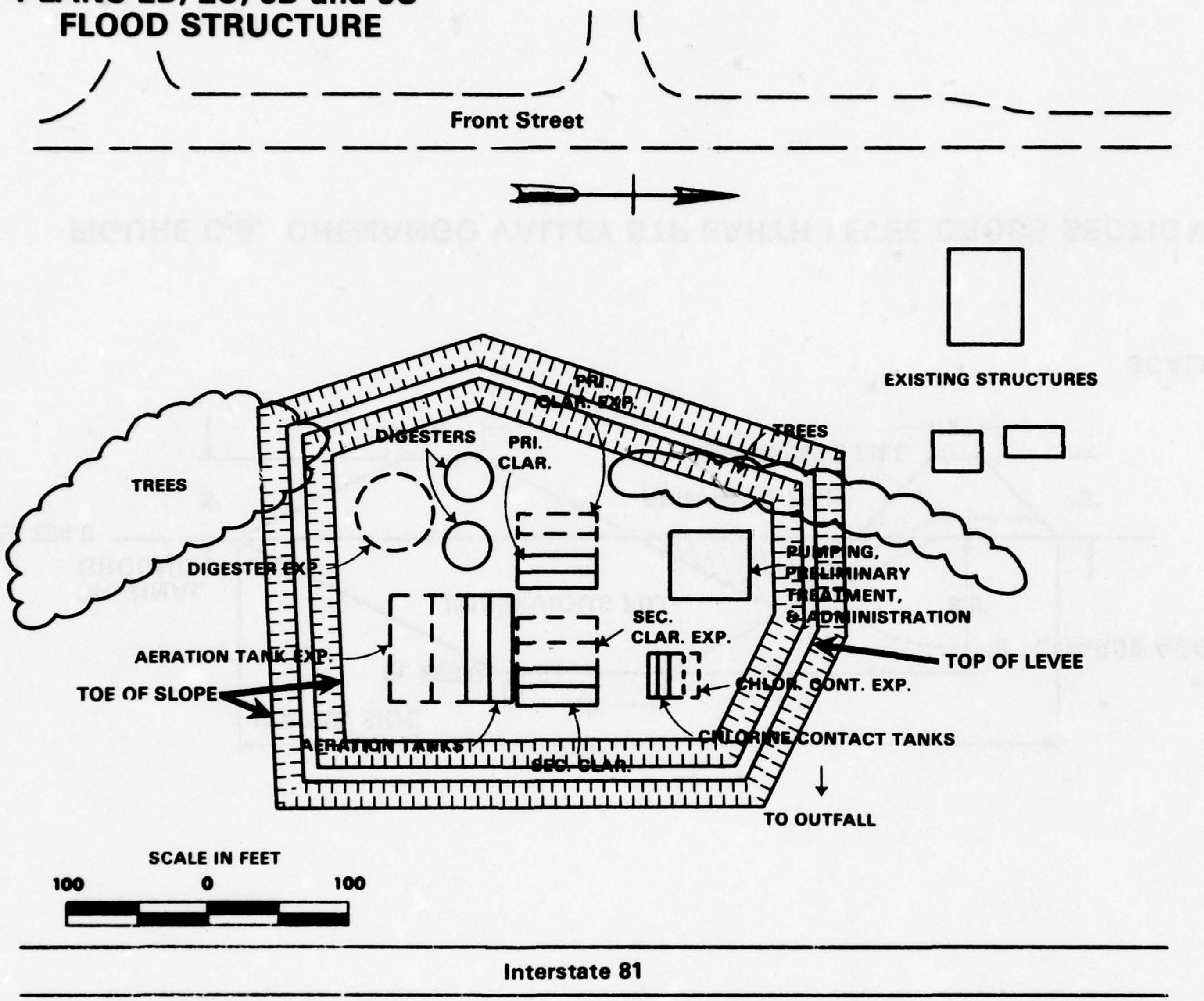


FIGURE C-10

TABLE C-6
CHENANGO VALLEY STP
FLOOD LEVEE CONSTRUCTION MATERIALS

ITEM	UNIT DESIGN VALUE [1] (PER LF)	TOTAL QUANTITIES PLAN 2B, 2C, 3B, 3C
Length of wall (LF)	-	1,160
Stripping (SY)	5.8	6,730
Excavation (CY)	4.2	4,880
Impervious Fill (CY)	9.7	11,260
Pervious Fill (CY)	1.25	1,450
Coarse Aggregate (CY)	0.96	1,115
Drainage Fill (CY)	1.1	1,280
Seeding & Sodding (SY)	4.2	4,880

[1] Based on typical Corps of Engineers' design criteria.

TABLE C-7
CHENANGO VALLEY STP
FLOOD LEVEE CONSTRUCTION BID SCHEDULE

ITEM	UNIT PRICE	UNIT	PLAN 2B, 2C, 3B, 3C QUANTITY	PLAN 2B, 2C, 3B, 3C COST
1. Stripping	0.25	SY	6,730	1,700
2. Excavation	6.00	CY	4,880	29,300
3. Impervious Fill	8.00	CY	11,260	90,100
4. Pervious Fill	2.50	CY	1,450	3,600
5. Coarse Aggregate	18.00	CY	1,115	20,100
6. Drainage Fill	13.00	CY	1,280	16,600
7. Seeding & Sodding	0.31	SY	4,880	1,500
	SUBTOTAL			$163,000
	ENGINEERING & CONTINGENCIES @ 30%			49,000
	TOTAL			$212,000

All costs on April 1975 dollars

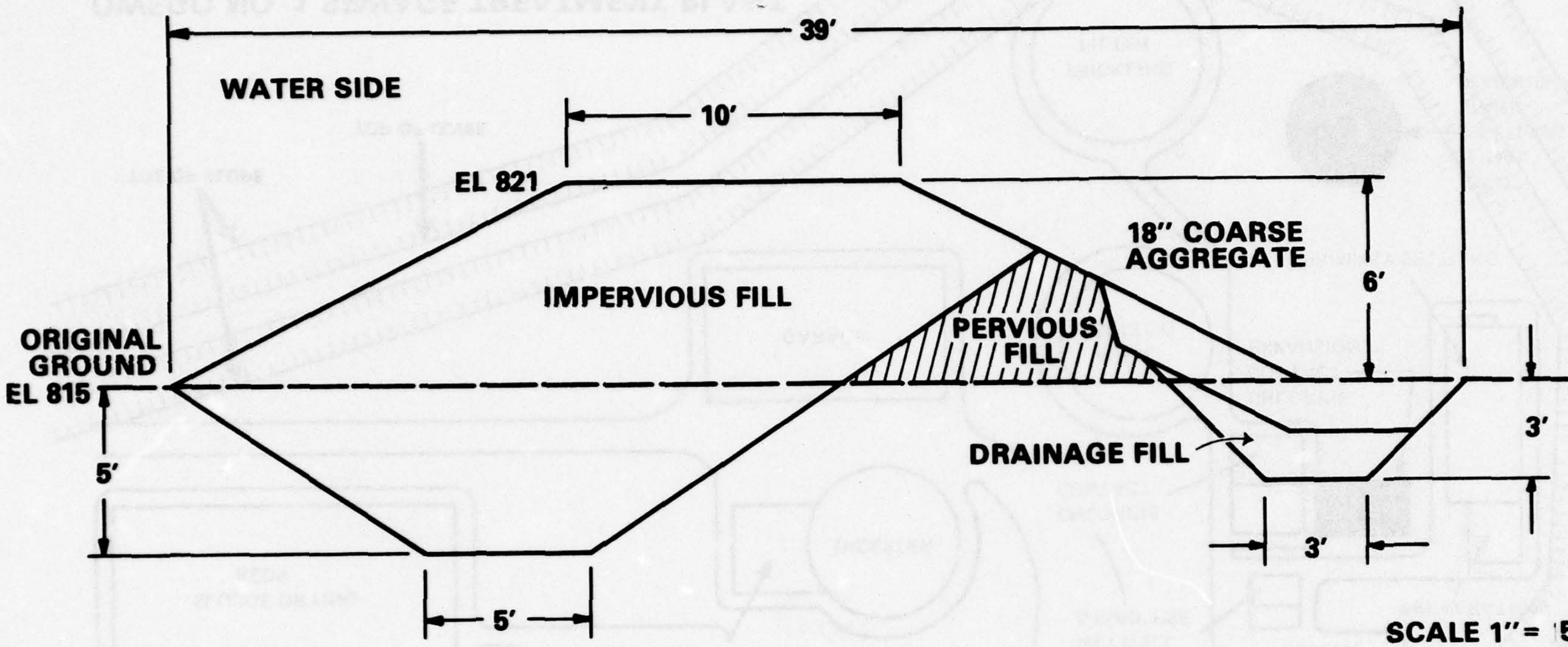
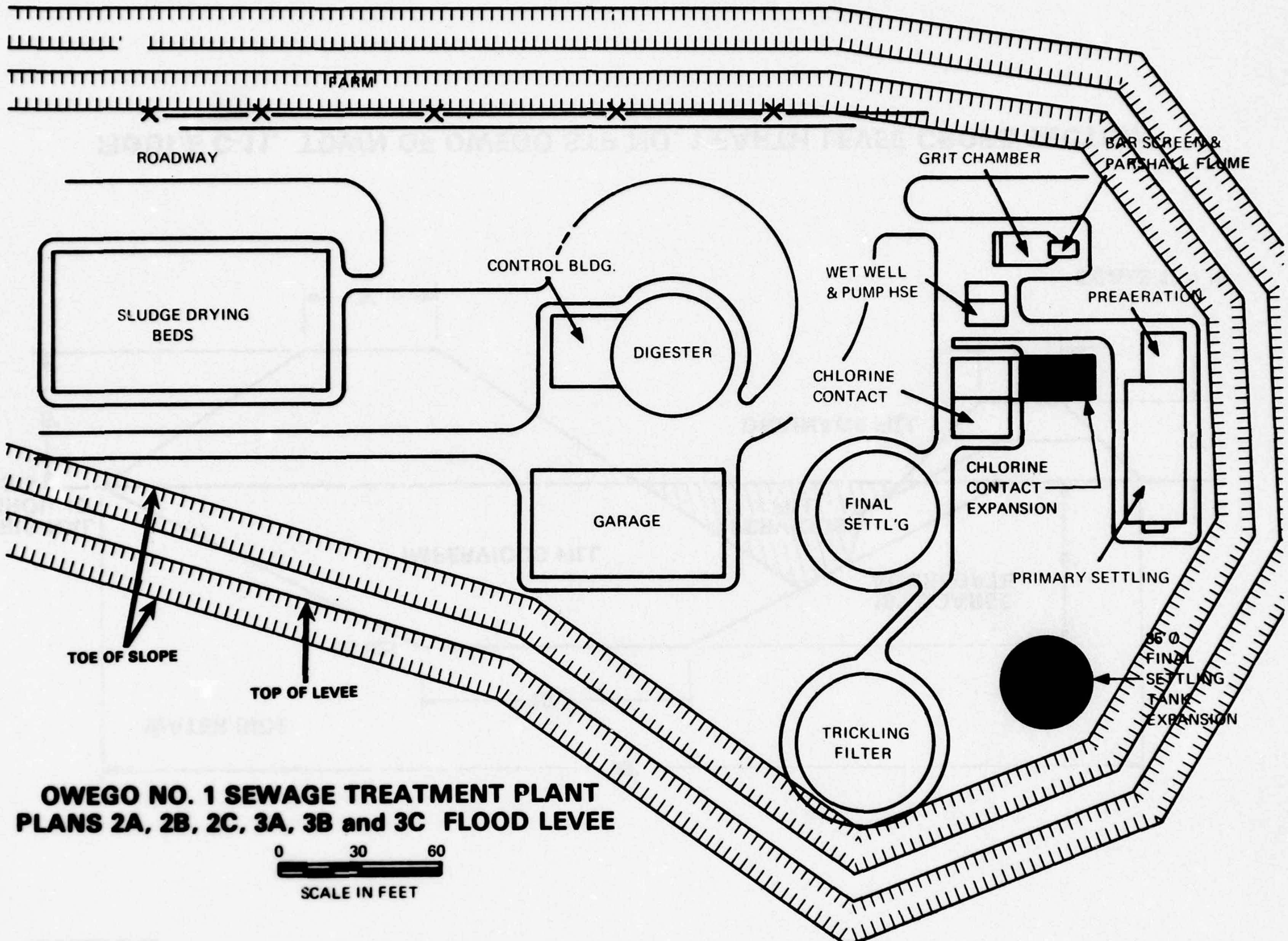


FIGURE C-11. TOWN OF OWEGO STP NO. 1 EARTH LEVEE CROSS SECTION



OWEGO NO. 1 SEWAGE TREATMENT PLANT
PLANS 2A, 2B, 2C, 3A, 3B and 3C FLOOD LEVEE

FIGURE C-12

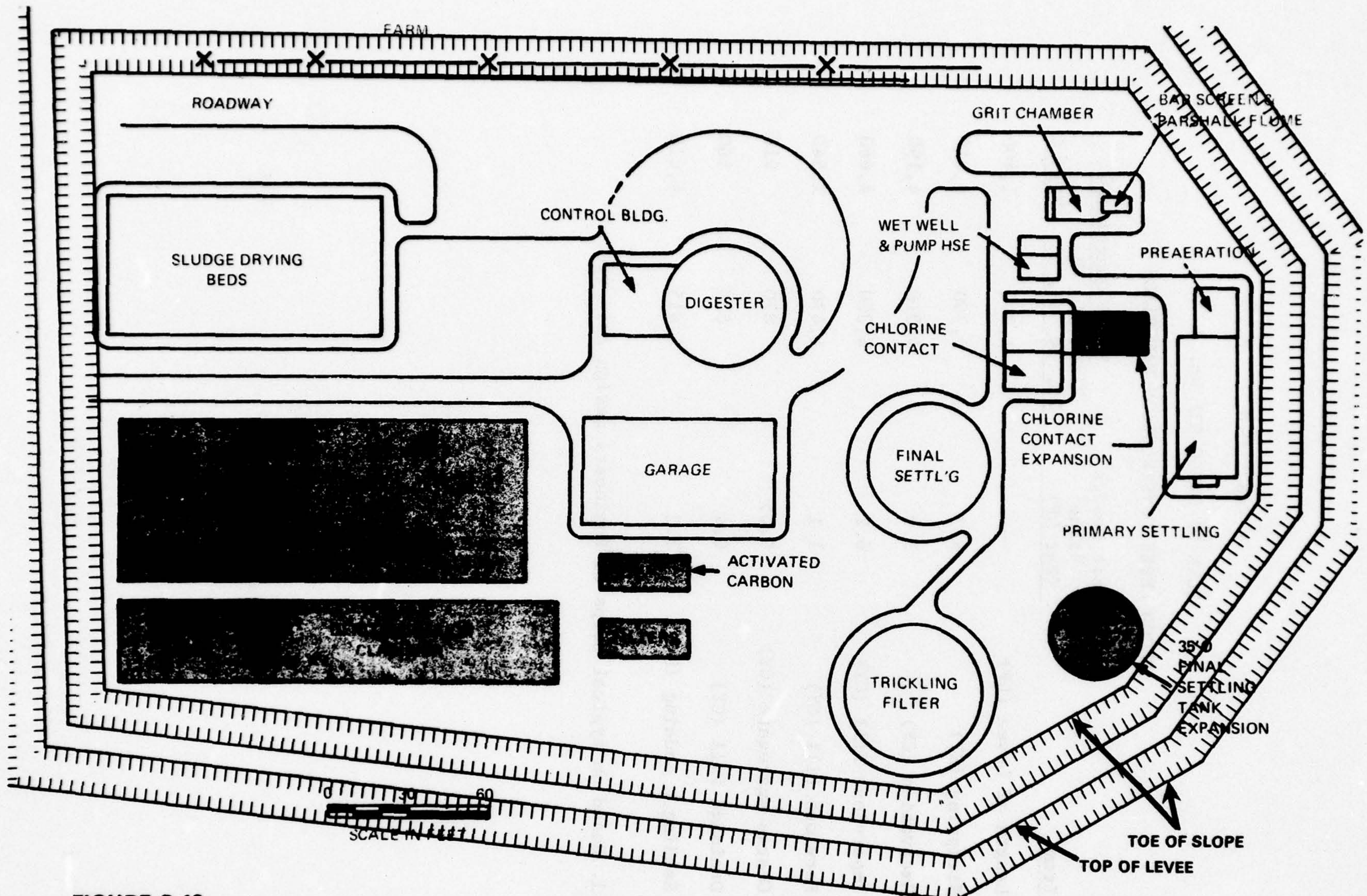


FIGURE C-13

BIO-AWT OWEGO NO. 1 SEWAGE TREATMENT PLANT
PLAN 4 FLOOD LEVEE

TABLE C-8

TOWN OF OWEGO STP No. 1

FLOOD LEVEE CONSTRUCTION MATERIALS

Item	Unit Design Value[1] (Per LF)	Total Quantities Plans 2A, 2B, 2C, 3A, 3B, 3C	Plan 4
Length of Levee (LF)	---	1,340	1,400
Stripping (SY)	4.3	5,750	6,010
Excavation (CY)	3.0	4,010	4,190
Impervious Fill (CY)	6.2	8,300	8,660
Pervious Fill (CY)	1.1	1,470	1,540
Coarse Aggregate (CY)	0.65	870	910
Drainage Fill (CY)	0.4	535	560
Seeding & Sodding (SY)	3.3	4,415	4,610

1. Based on typical Corps of Engineers design criteria.

TABLE C-9
TOWN OF OWEGO STP No. 1
FLOOD LEVEE CONSTRUCTION BID SCHEDULE

ITEM	UNIT PRICE ($)	UNIT	PLAN 2A, 2B, 2C, 3A, 3B, 3C QUANTITY	PLAN 2A, 2B, 2C, 3A, 3B, 3C COST $	PLAN 4 QUANTITY	PLAN 4 COST $
1. Stripping	0.25	SY	5,750	1,400	6,010	1,500
2. Excavation	6.00	CY	4,010	24,100	4,190	25,100
3. Impervious Fill	8.00	CY	8,300	66,400	8,660	69,300
4. Pervious Fill	2.50	CY	1,470	3,700	1,540	3,800
5. Coarse Aggregate	20.00	CY	870	17,400	910	18,200
6. Drainage Fill	13.00	CY	535	7,000	560	7,300
7. Seeding & Sodding	0.31	SY	4,415	1,400	4,610	1,400
	SUBTOTAL			$121,000		$127,000
	ENGINEERING & CONTINGENCIES @ 30%			37,000		38,000
	TOTAL $			$158,000		$165,000

All costs on April 1975 dollars

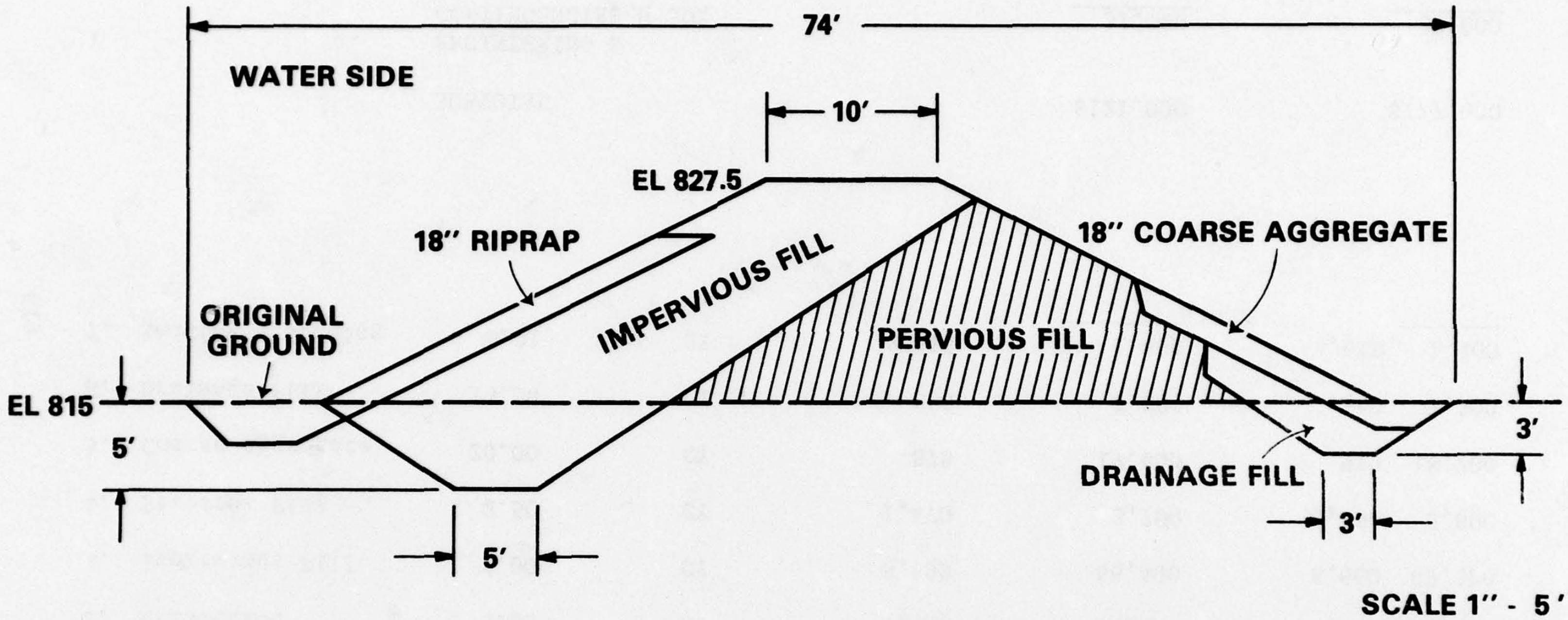


FIGURE C-14. TOWN OF OWEGO STP NO. 2 EARTH LEVEE CROSS SECTION

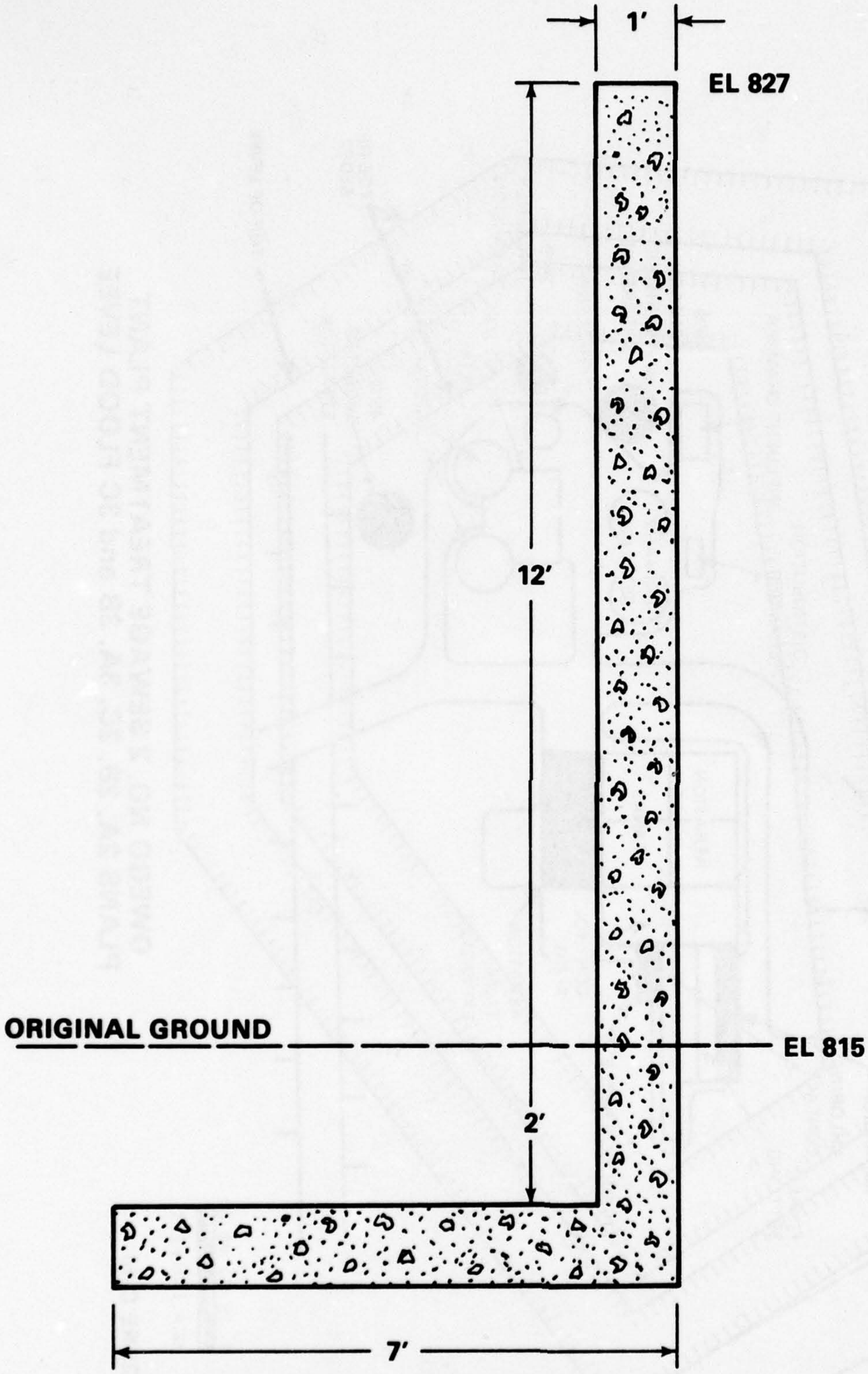


FIGURE C-15. TOWN OF OWEGO STP NO. 2 CONCRETE ELL WALL CROSS SECTION

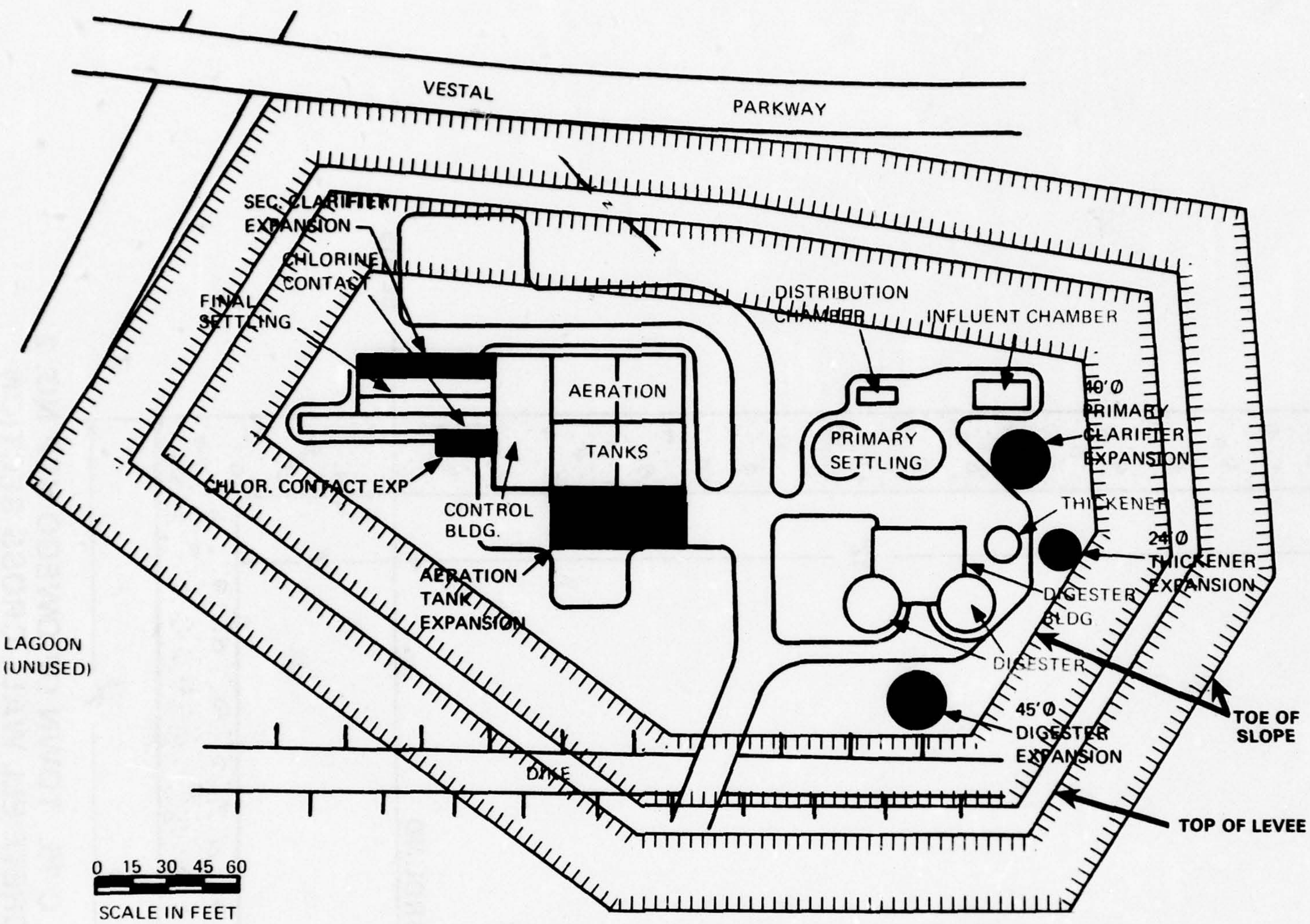


FIGURE C-16

**OWEGO NO. 2 SEWAGE TREATMENT PLANT
PLANS 2A, 2B, 2C, 3A, 3B and 3C FLOOD LEVEE**

BIO-AWT OWEGO NO. 2 SEWAGE TREATMENT PLANT
PLAN 4 FLOOD WALL & LEVEE

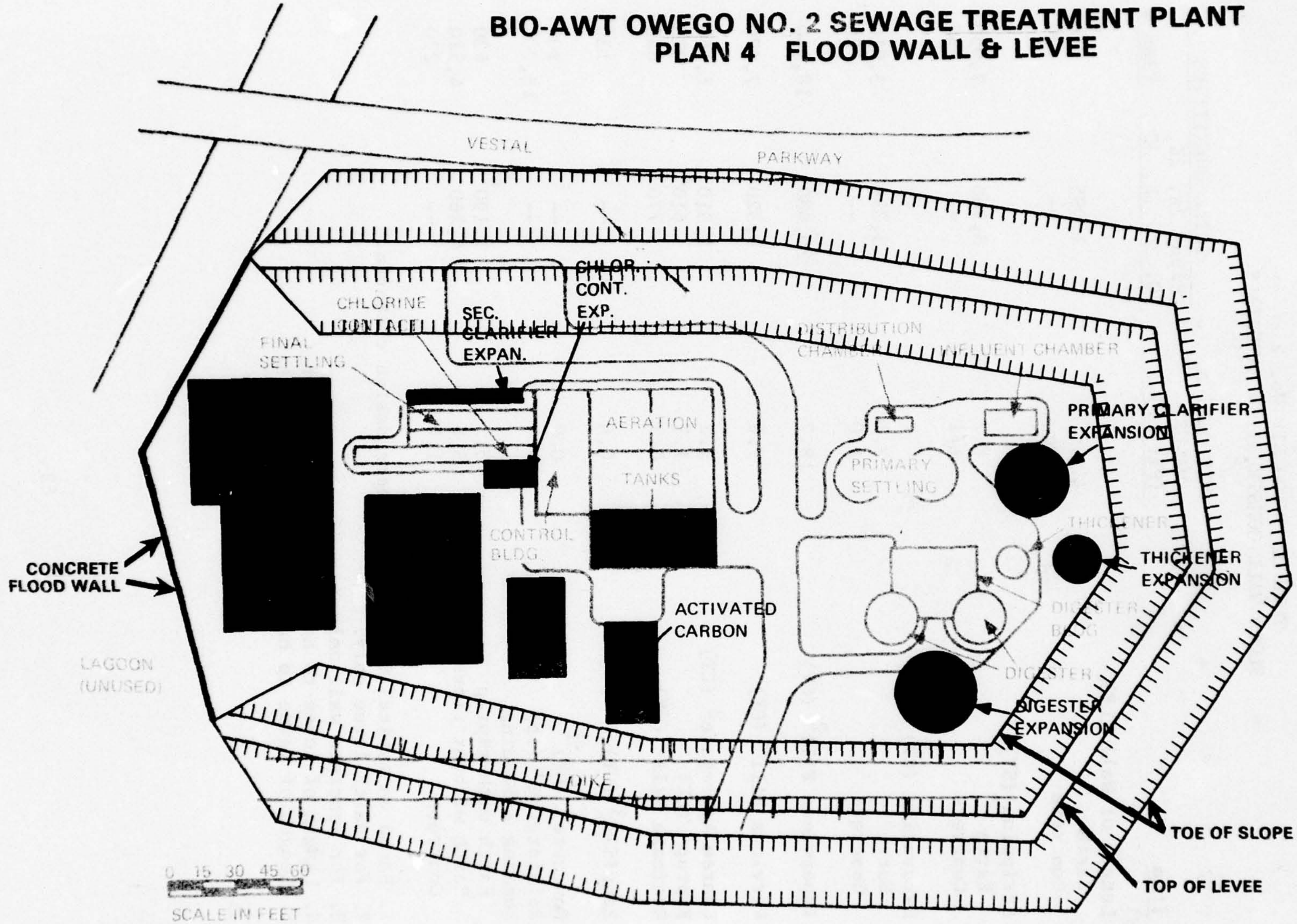


FIGURE C-17

TABLE C-10

TOWN OF OWEGO STP No. 2
FLOOD WALL CONSTRUCTION MATERIALS

Item	UNIT DESIGN VALUE[1] (Per LF)	TOTAL QUANTITIES Plans 2A, 2B, 2C, 3A, 3B, 3C	Plan 4
Length of Wall (LF)			
Earth	--	1,095	970
Concrete	--	--	220
Stripping (SY)			
Earth	8.2	8,980	7,950
Concrete	1.1	--	240
Excavation (CY)			
Earth	3.1	3,390	3,010
Concrete	1.0	--	220
Impervious Fill[2] (CY)	18.7	20,480	18,140
Pervious Fill[2] (CY)	7.6	8,320	7,370
Coarse Aggregate[2] (CY)	1.2	1,310	1,164
Riprap[2] (CY)	1.7	610[4]	320[5]
Drainage Fill[2] (CY)	0.7	770	680
Backfill[3] (CY)	0.6	--	130
Concrete[3] (CY)	0.9	--	200
Reinforcing[3] (lb)	60	--	13,200
Seeding & Sodding			
Earth with riprap	3.3	1,190[4]	630
Earth without riprap	5.8	4,260	4,520
Concrete	1.0	--	220

1. Based on typical Corps of Engineers design criteria.
2. For earth levee only.
3. For concrete wall only.
4. Length of levee to have riprap is 360 feet.
5. Length of levee to have riprap is 190 feet.

TABLE C-11

TOWN OF OWEGO STP No. 2 -- FLOOD WALL CONSTRUCTION BID SCHEDULE

Item	UNIT PRICE ($)	UNIT	PLANS 2A, 2B, 2C, 3A, 3B, 3C Quantity	PLANS 2A, 2B, 2C, 3A, 3B, 3C Cost $	PLAN 4 Quantity	PLAN 4 Cost $
1. Stripping	0.25	SY	8,980	2,200	8,200	2,000
2. Excavation: Structural	10.50	CY	--	--	220	2,300
Unclassified	6.00	CY	3,390	20,400	3,010	18,000
3. Impervious Fill	8.00	CY	20,480	163,800	18,140	145,100
4. Pervious Fill	2.50	CY	8,320	20,800	7,370	18,400
5. Coarse Aggregate	18.00	CY	1,310	23,700	1,160	21,000
6. Riprap -- 18"	35.00	CY	610	21,400	320	11,300
7. Drainage Fill	13.00	CY	770	10,000	680	8,800
8. Backfill	8.00	CY	--	--	130	1,100
9. Concrete	190.00	CY	--	--	200	37,600
10. Reinforcing	0.50	lb.	--	--	13,200	6,600
11. Seeding & Sodding	0.31	SY	5,450	1,700	5,370	1,700
12. Closure Structure (Ht.-5 ft.)	15,000	EA	1	15,000	1	15,000
			Sub-Total	$279,000		$289,000
Engineering & Contingencies @ 30%				84,000		87,000
			TOTAL	$363,000		$376,000

All costs in April 1975 dollars.

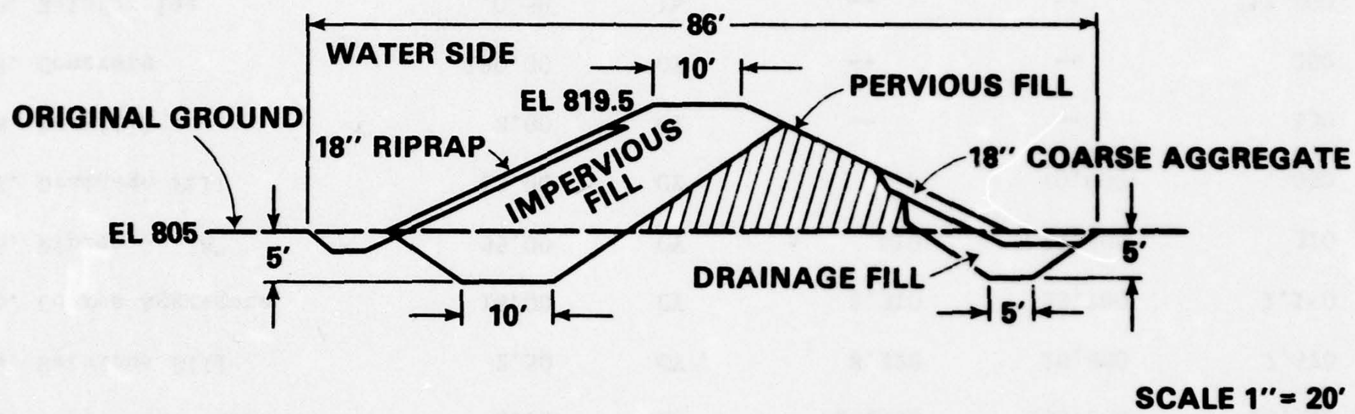


FIGURE C-18. OWEGO VILLAGE STP EARTH LEVEE CROSS SECTION

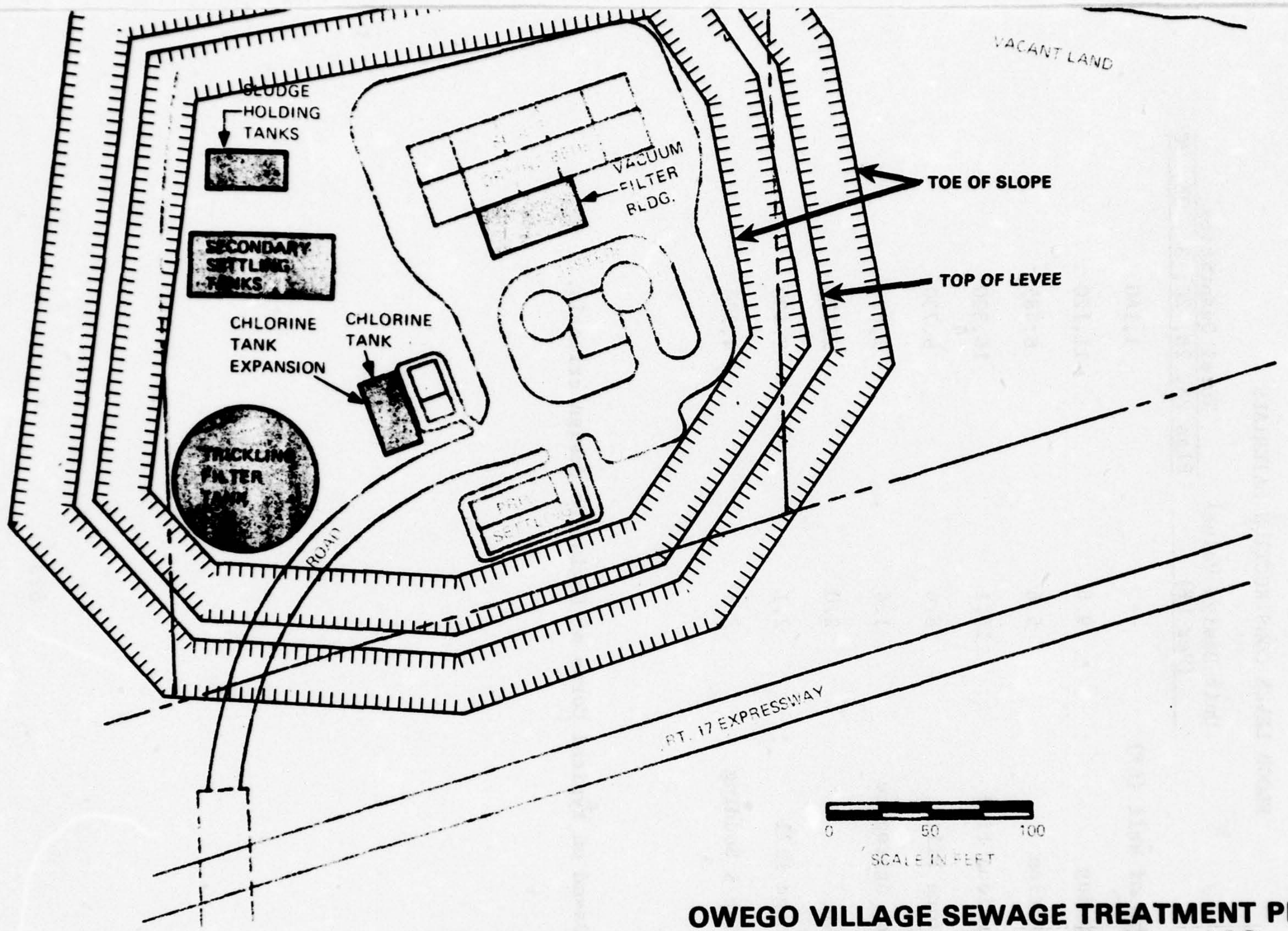


FIGURE C-19

OWEGO VILLAGE SEWAGE TREATMENT PLANT
PLANS 2A, 2B, 2C, 3A, 3B and 3C
FLOOD LEVEE

TABLE C-12

VILLAGE OF OWEGO STP

FLOOD LEVEE CONSTRUCTION MATERIALS

Item	Unit Design Value[1] (Per LF)	Total Quantities Plan 2A, 2B, 2C, 3A, 3B, 3C
Length of Wall (LF)	--	1,160
Stripping	9.6	11,120
Excavation	5.6	6,485
Impervious Fill	14.1	16,330
Pervious Fill	8.4	9,730
Coarse Aggregate	1.4	1,620
Riprap	2.0	2,320
Drainage Fill	2.1	2,430
Seeding & Sodding	3.6	4,200

1. Based on typical Corps of Engineers design criteria.

TABLE C-13
OWEGO VILLAGE STP
FLOOD LEVEE CONSTRUCTION BID SCHEDULE

ITEM	UNIT PRICE	UNIT	PLAN 2A, 2B, 2C, 3A, 3B, 3C QUANTITY	COST $
1. Stripping	0.25	SY	11,120	2,800
2. Excavation	6.00	CY	6,485	38,900
3. Impervious Fill	8.00	CY	16,330	130,600
4. Pervious Fill	2.50	CY	9,730	24,300
5. Coarse Aggregate	18.00	CY	1,620	29,200
6. Riprap - 18"	35.00	CY	2,320	81,100
7. Drainage Fill	13.00	CY	2,430	31,600
8. Seeding & Sodding	0.31	SY	4,170	1,300
9. Closure Structure	15,000	EA	1	15,000
	SUBTOTAL			$355,000
	ENGINEERING & CONTINGENCIES @ 30%			106,000
	TOTAL $			$461,000

All costs on April 1975 dollars

TABLE C-14

COST PER LINEAR FOOT OF FLOODWALL*
($)

STP	WASTEWATER MANAGEMENT PLAN 1	2A	2B	2C	3A	3B	3C	4
Binghamton-Johnson City	---	180	180	180	180	180	180	180
Endicott	---	510	510	510	510	510	510	510
Chenango Valley	---	---	180	180	---	180	180	---
Owego No. 1	---	120	120	120	120	120	120	120
Owego No. 2	---	330	330	330	330	330	330	330
Owego Village	---	400	400	400	400	400	400	---

*April 1975 dollars.

TABLE C-15

LENGTH OF FLOOD WALL (Linear Feet)

STP	WASTEWATER MANAGEMENT PLAN							
	1	2A	2B	2C	3A	3B	3C	4
Binghamton-Johnson City (Concrete ELL Wall)	---	2,860	2,820	2,820	2,850	2,850	2,850	2,925
Endicott (Earth Levee with Riprap)	---	3,100	3,100	3,100	3,100	3,100	3,100	3,240
Chenango Valley (Earth Levee)	---	---	1,160	1,160	---	1,160	1,160	---
Owego No. 1 (Earth Levee)	---	1,340	1,340	1,340	1,340	1,340	1,340	1,400
Owego No. 2 (Earth Levee with Riprap Includes 220 foot concrete wall in Plan 4)	---	1,095	1,095	1,095	1,095	1,095	1,095	1,190
Owego Village (Earth Levee with Riprap)	---	1,160	1,160	1,160	1,160	1,160	1,160	---

TABLE C-16

COST OF FLOOD PROOFING STP's[1]
($1,000)

STP	WASTEWATER MANAGEMENT PLAN[2]							
	1	2A	2B	2C	3A	3B	3C	4
Binghamton-Johnson City	---	509	502	502	507	507	507	519
Endicott	---	1,587	1,587	1,587	1,587	1,587	1,587	1,656
Chenango Valley	---	---	212	212	---	212	212	---
Owego No. 1	---	158	158	158	158	158	158	165
Owego No. 2	---	363	363	363	363	363	363	376
Owego Village	---	461	461	461	461	461	461	---
Total	---	3,078	3,283	3,283	3,076	3,288	3,288	2,716

1. April 1975 Dollars.

2. Note: Costs in this Table reflect only the costs of floodproofing.

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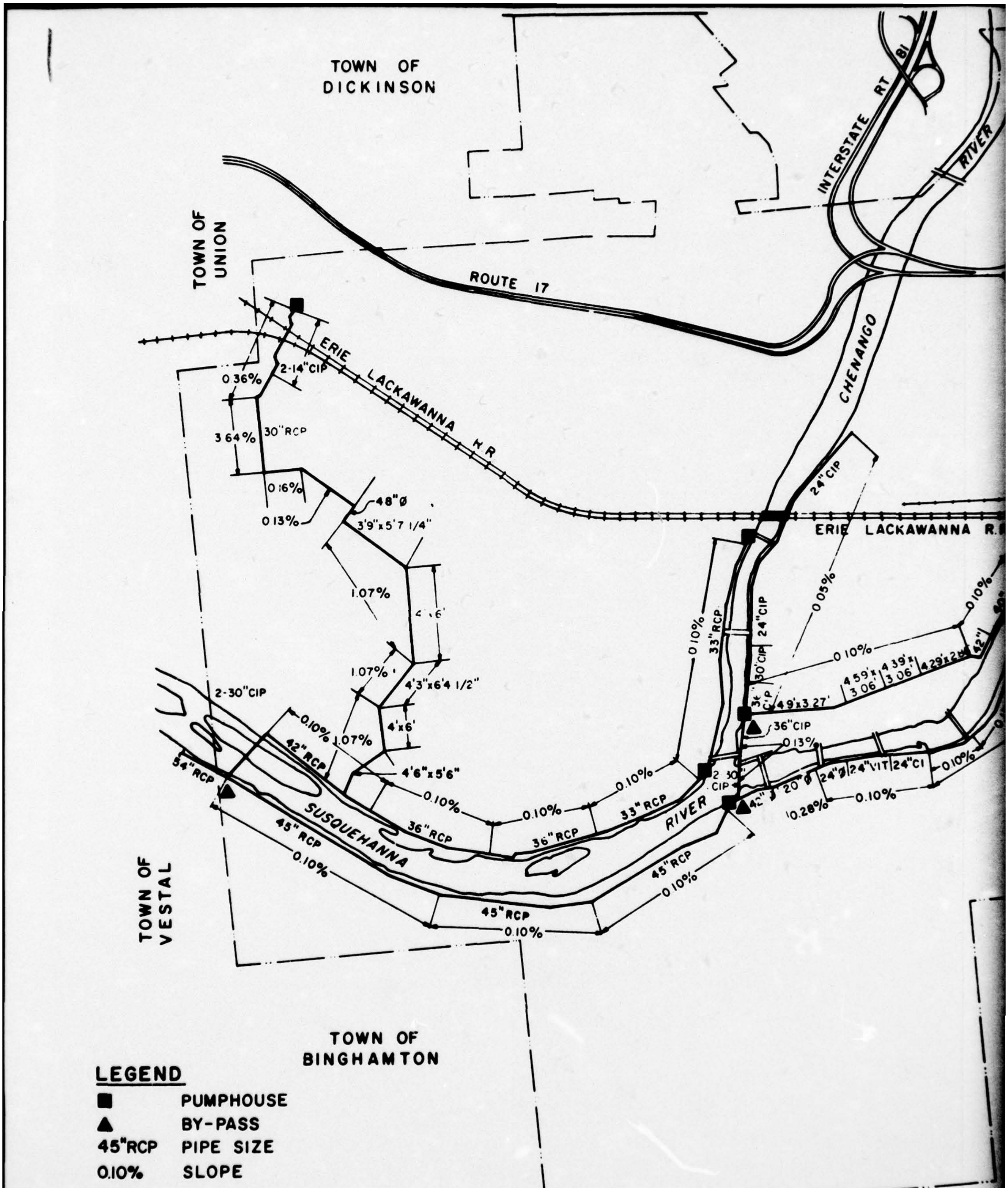
LIST OF ABBREVIATIONS

AAC	Average Annual Cost
AWT	Adverage Waste Treatment
BAQMA	Binghamton Air Quality Maintenance Area
BAT	Best Available Technology Currently Available
BCHD	Broome County Health Department
BCSA	Broome County Sewer Agency
B-JC	Binghamton-Johnson City
BJCJSB	Binghamton-Johnson City Joint Sewer Board
BOD	Biochemical Oxygen Demand
BPT	Best Practicable Technology Economically Achievable
BPWTT	Best Practicable Waste Treatment Technology
CAC	Citizens Advisory Committee
CFS	cubic feet per second
COD	Chemical Oxygen Demand
COE	US Army Corps of Engineers
DO	Dissolved Oxygen
DOT	Department of Transportation
EPA	Environmental Protection Agency
GPCD	gallons per capita per day
GPD	gallons per day
GPM	gallons per minute
HUD	US Department of Housing and Urban Development
ISMG	Interagency Study Management Group
LMS	Lawler, Matuskey, and Skelly Engineers
MA-7-CD-10	minimum average seven consecutive day flow occurring once in ten years
mgd	million gallons per day
mg/l	milligrams per liter
ml	mililiters
MLVSS	Mixed Liquor Volatile Suspended Solids
MPN	Most Probable Number
NOD	Nitrogenous Oxygen Demand
NPDES	National Pollutant Discharge Elimination System
NYS	New York State
NYSDEC	New York State Department of Environmental Conservation
O&M	Operation & Maintenance
P/C	physical/chemical
POS	Plan of Study
PW	Present Worth
QLM	Quirk, Lawler, and Matusky Engineers
SCS	US Department of Agriculture Soil Conservation Service
SRBC	Susquehanna River Basin Commission
SS	Suspended Solids
STERPB	Southern Tier East Regional Planning Board
STP	Sewage Treatment Plant
SWMM	Storm Water Management Model
TAC	Technical Advisory Committee
TCCS	Tioga County Comprehensive Sewage Study
WPCP	Water Pollution Control Plant



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TOWN OF UNION
TOWN OF VESTAL
TOWN OF BINGHAMTON
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ERIE LACKAWANNA R.R
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0.16%
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1.07%
1.07%
4'3"x6'4 1/2"
4'x6'
1.07%
4'6"x5'6"
2-30"CIP
0.10%
42"RCP
54"RCP
36"RCP
0.10%
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36"RCP
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0.10%
33"RCP
24"CIP
30"CIP
0.10%
4.59'x 3.06'
4.39'x 3.06'
4.29'x 2.8'
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4.9'x 3.27'
36"CIP
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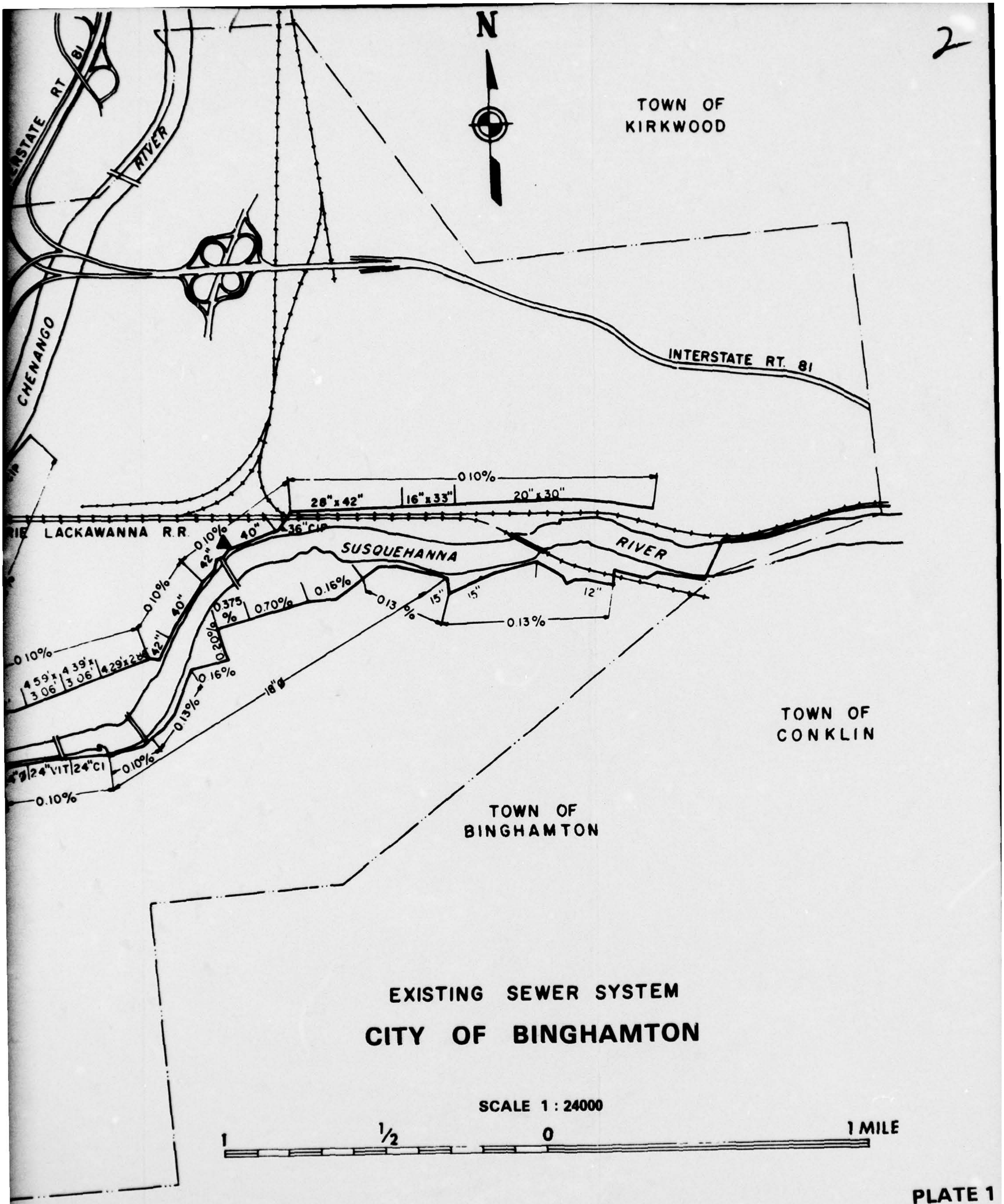
0.10% SLOPE



EXISTING SEWER SYSTEM

CITY OF BINGHAMTON

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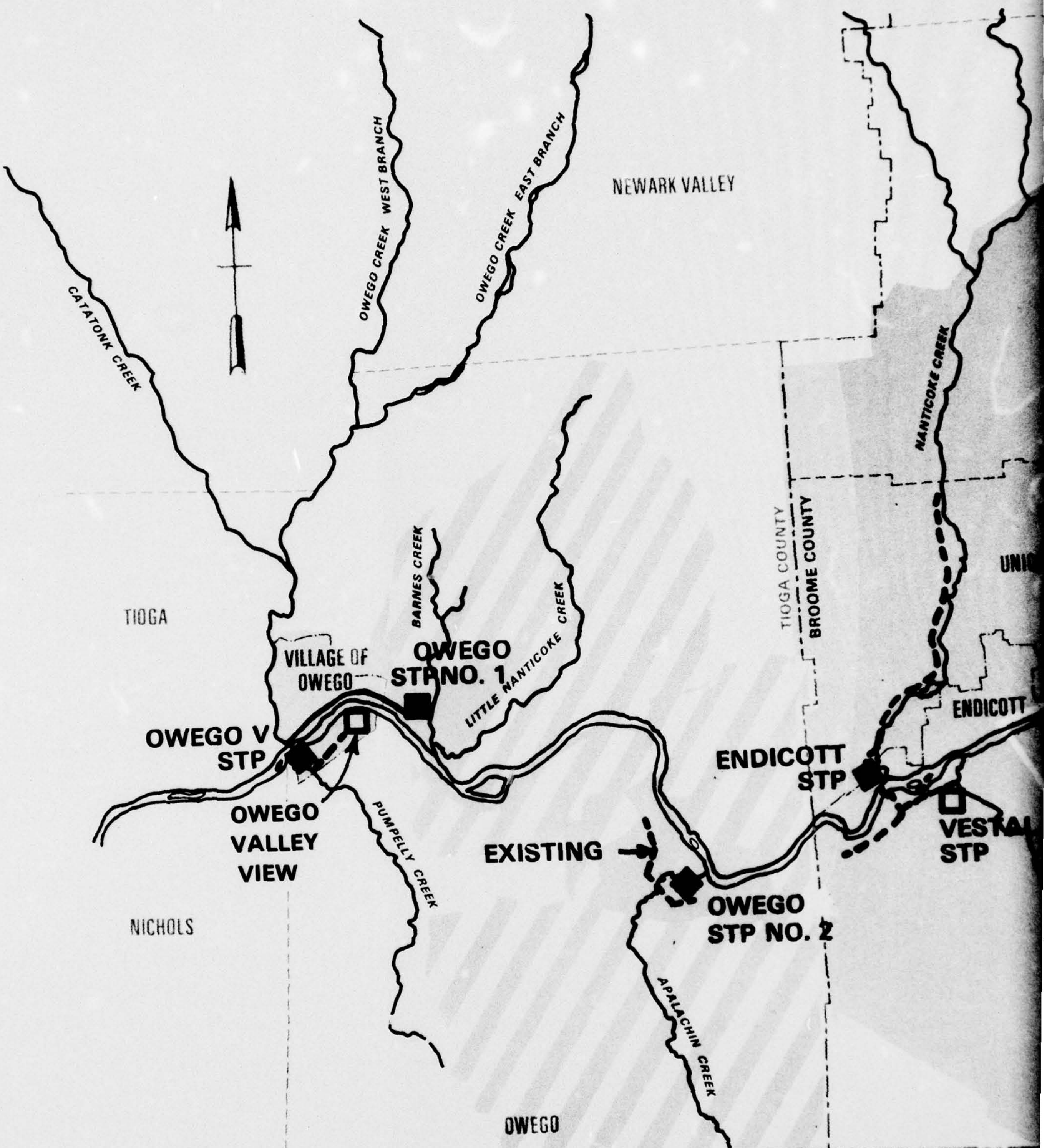
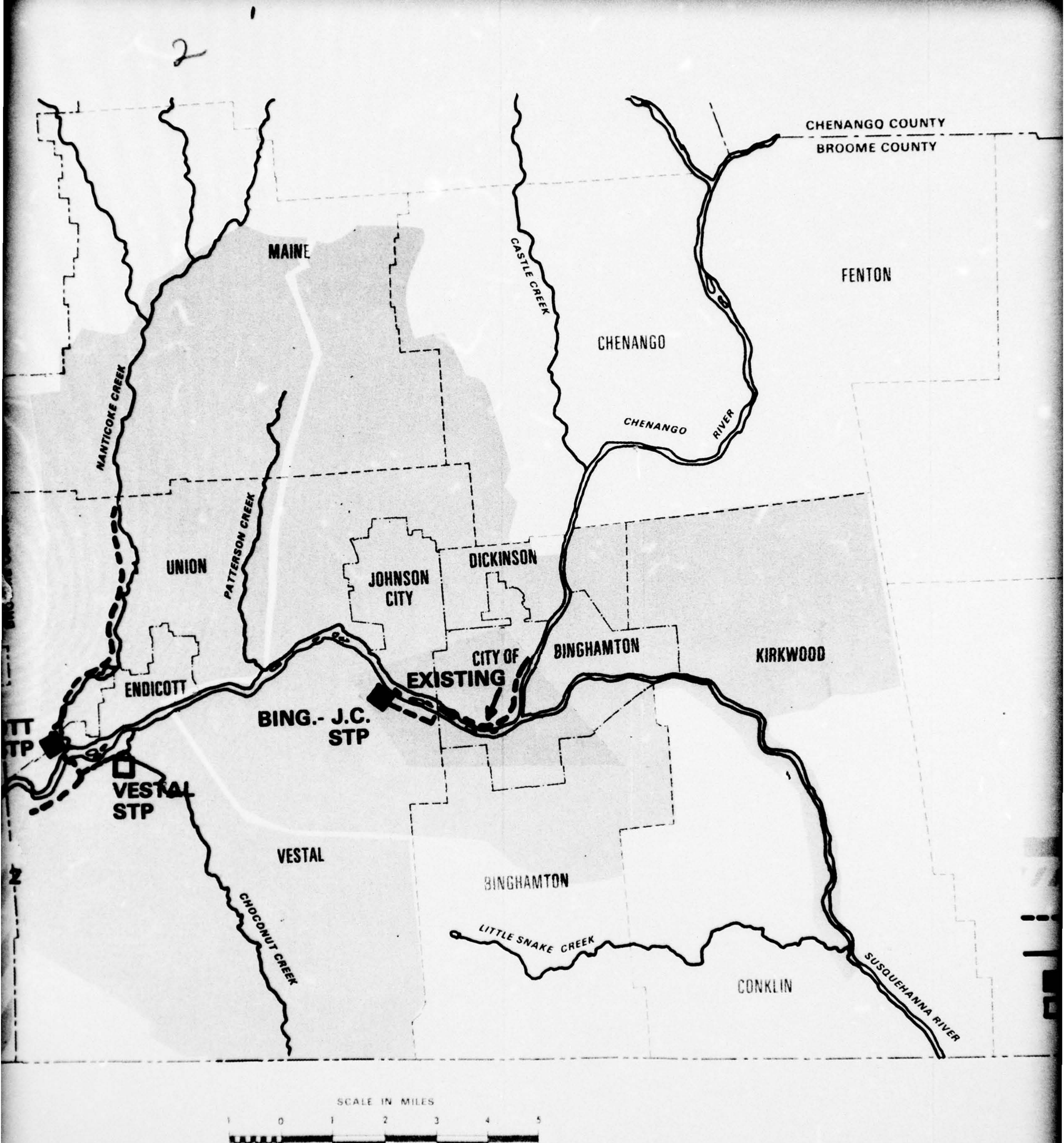
PLATE 1

BASELINE PROFILE

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DICKINSON
JOHNSON CITY
CITY OF BINGHAMTON
KIRKWOOD
EXISTING
ENDICOTT
BING.- J.C. STP
VESTAL STP
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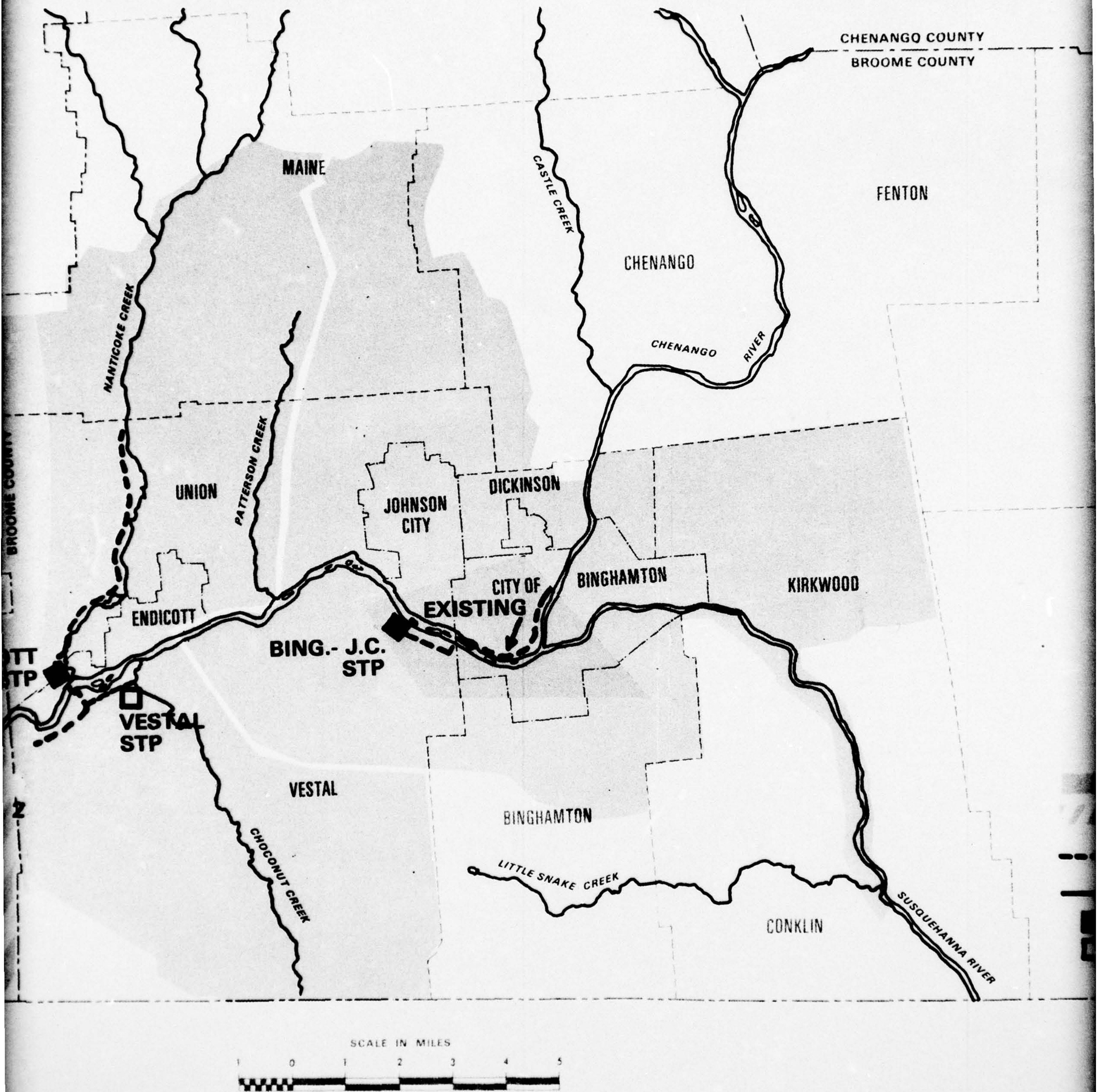

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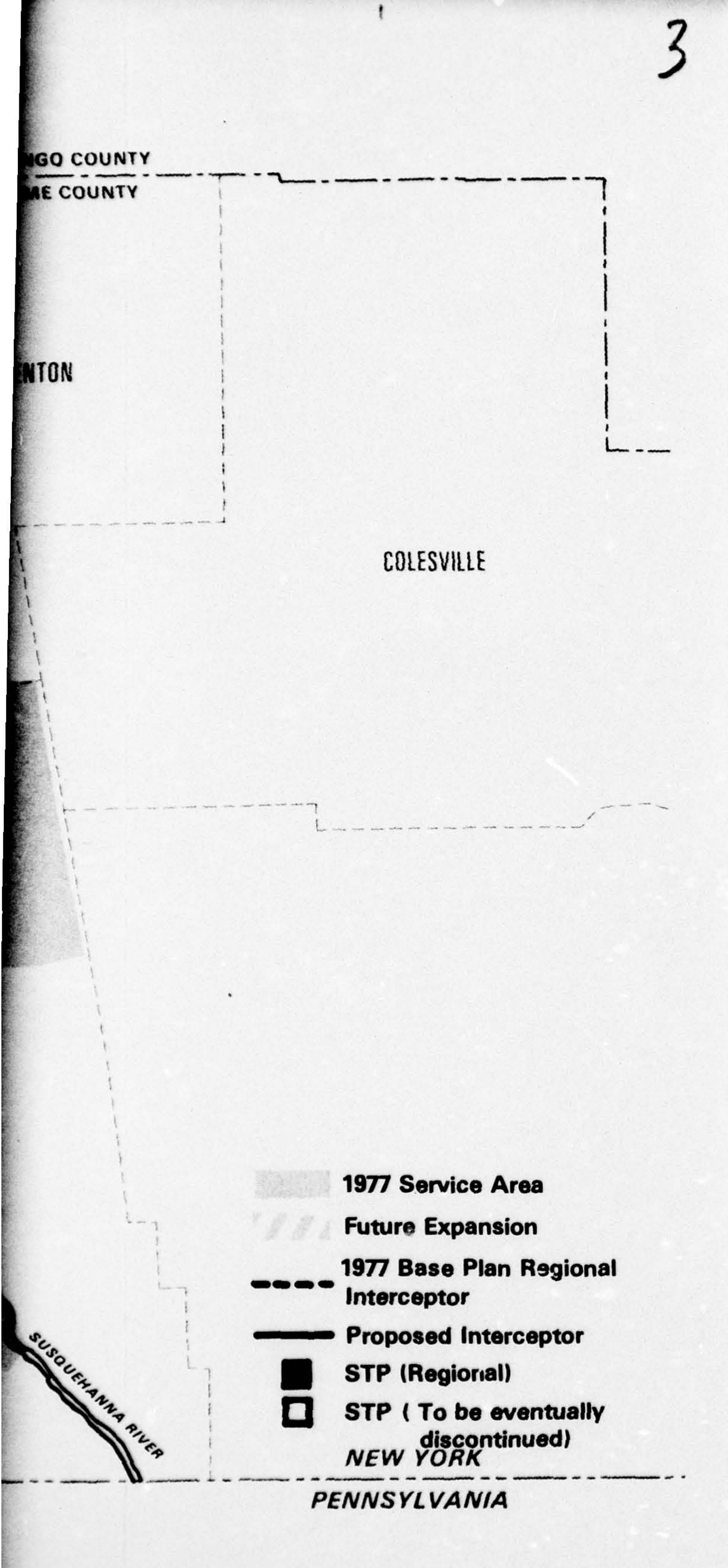
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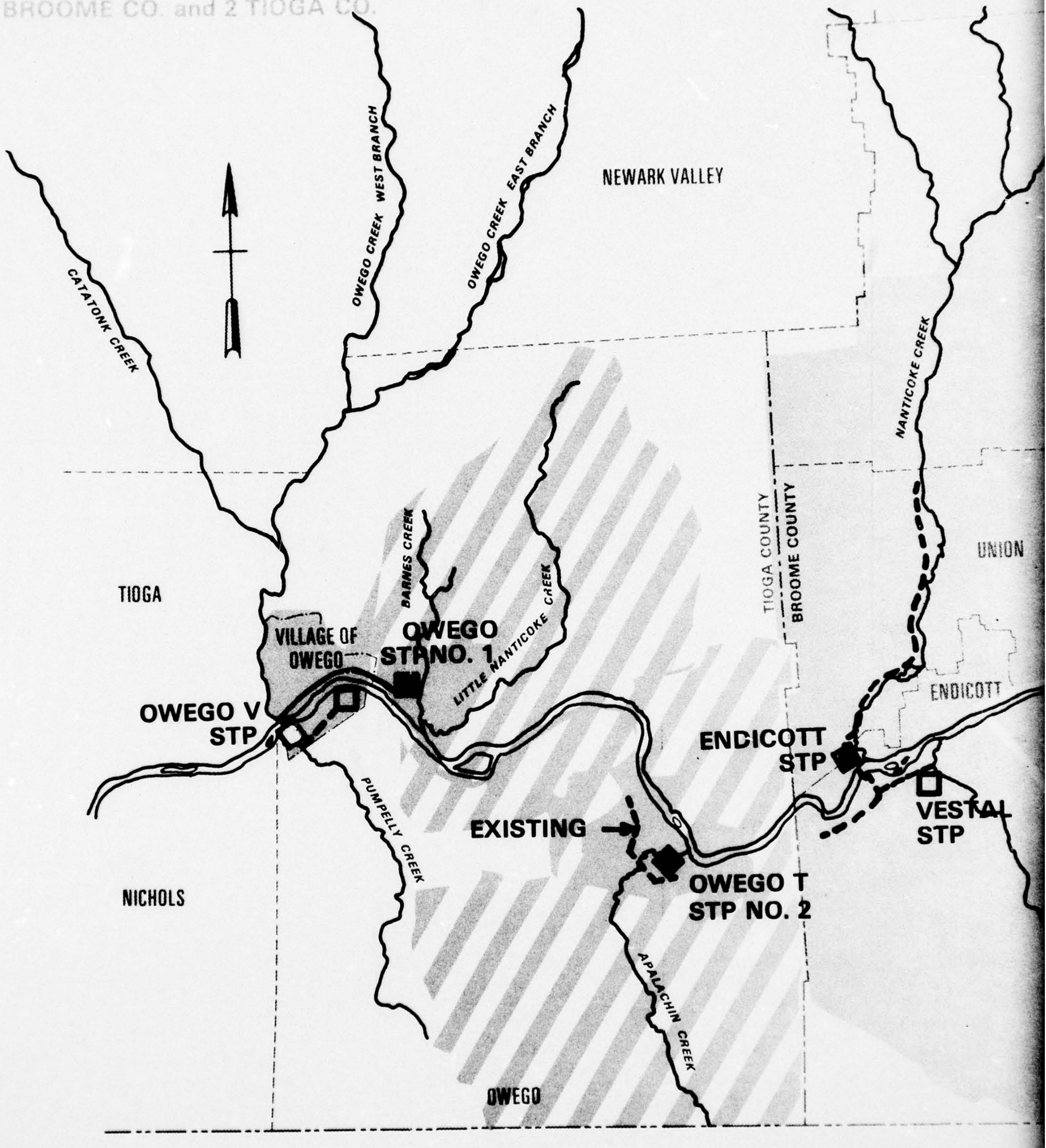
1977 Base Plan Regional Interceptor
Proposed Interceptor
STP (Regional)
STP (To be eventually discontinued)
SUSQUEHANNA RIVER
NEW YORK
PENNSYLVANIA

PLATE 2

1

FOUR REGIONAL TREATMENT PLANTS

2 BROOME CO. and 2 TIOGA CO.



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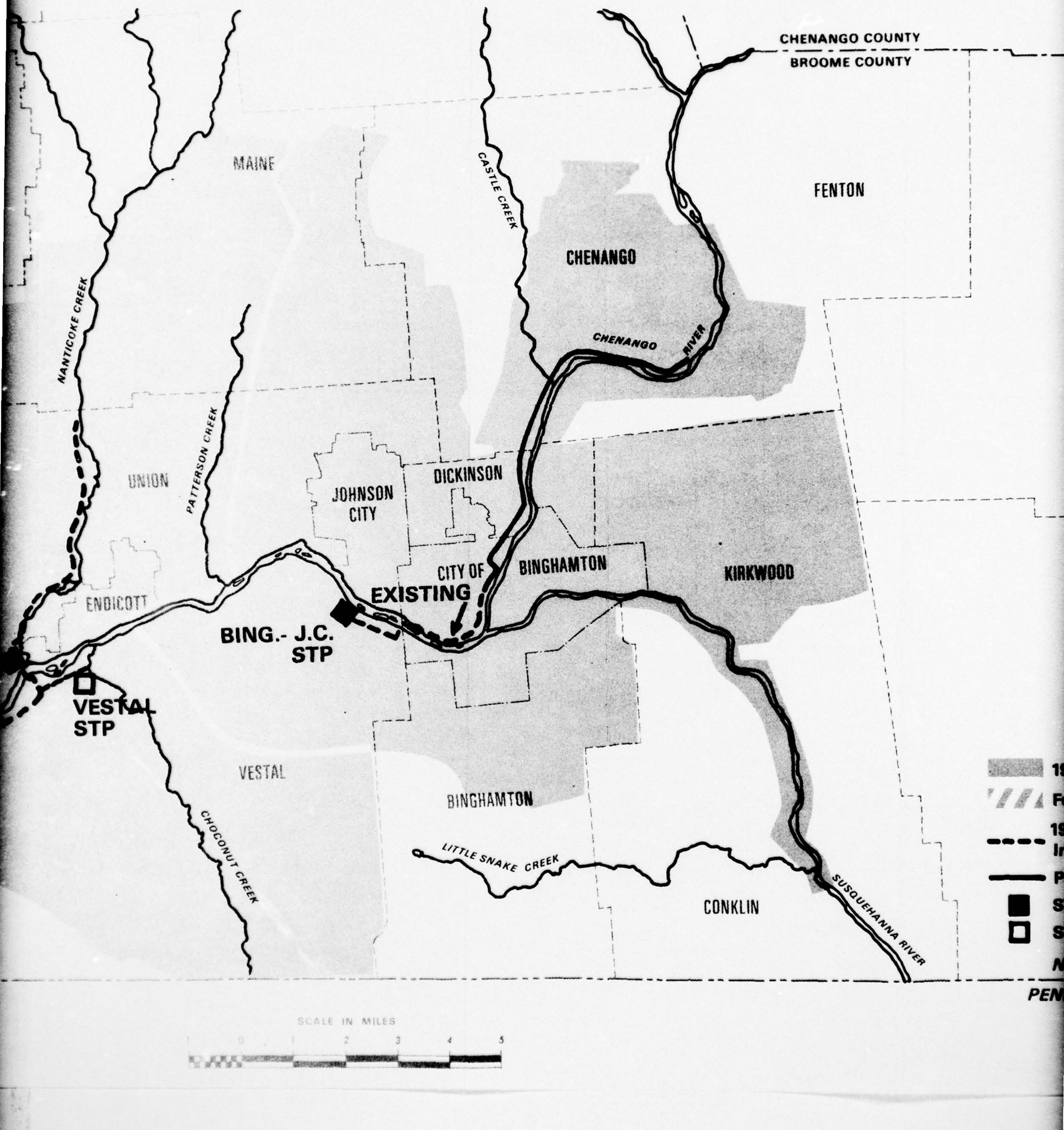
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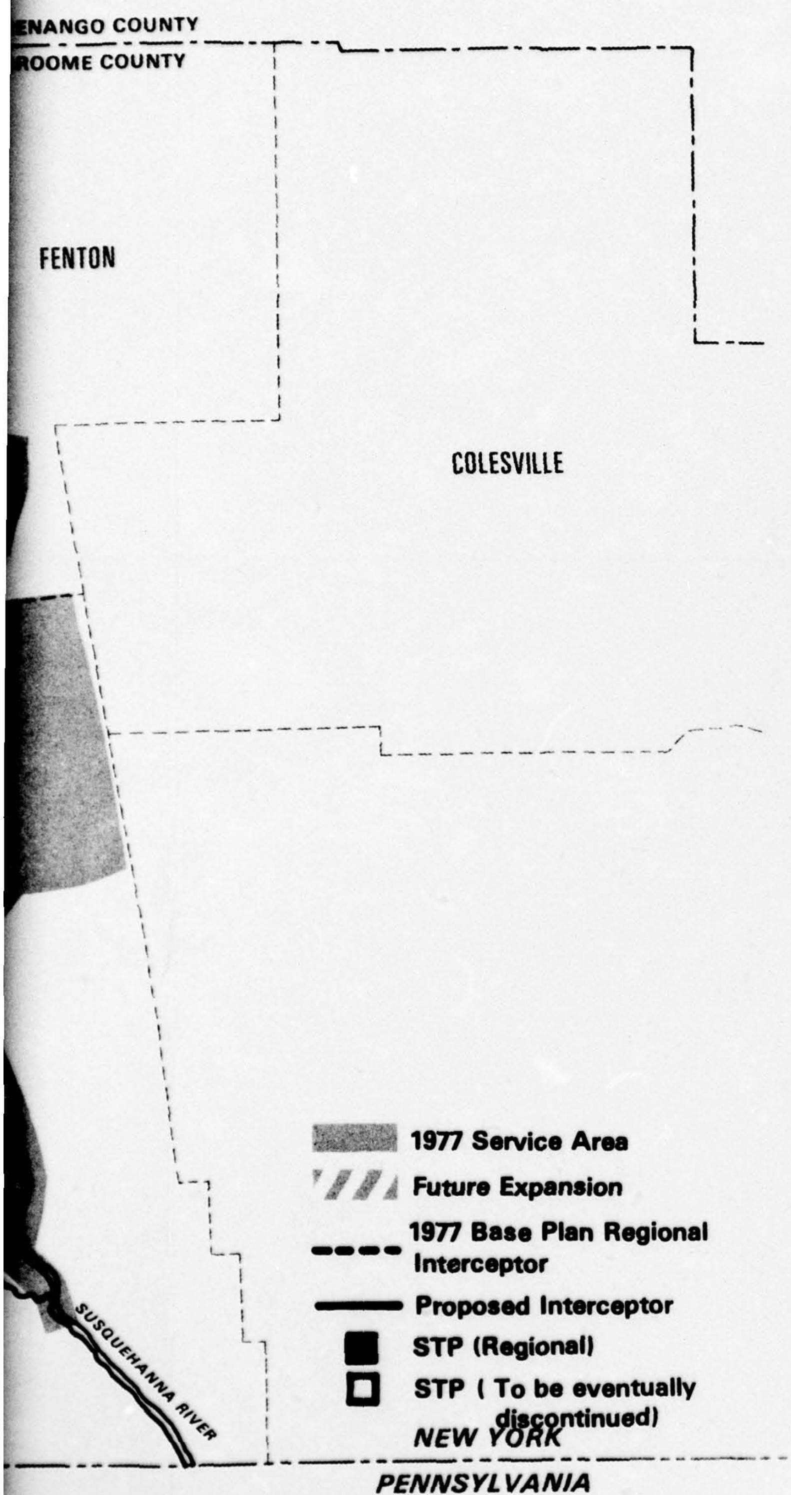

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NEW YORK
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PLATE 3

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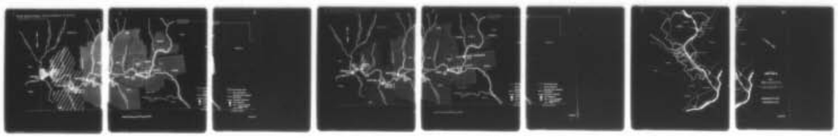

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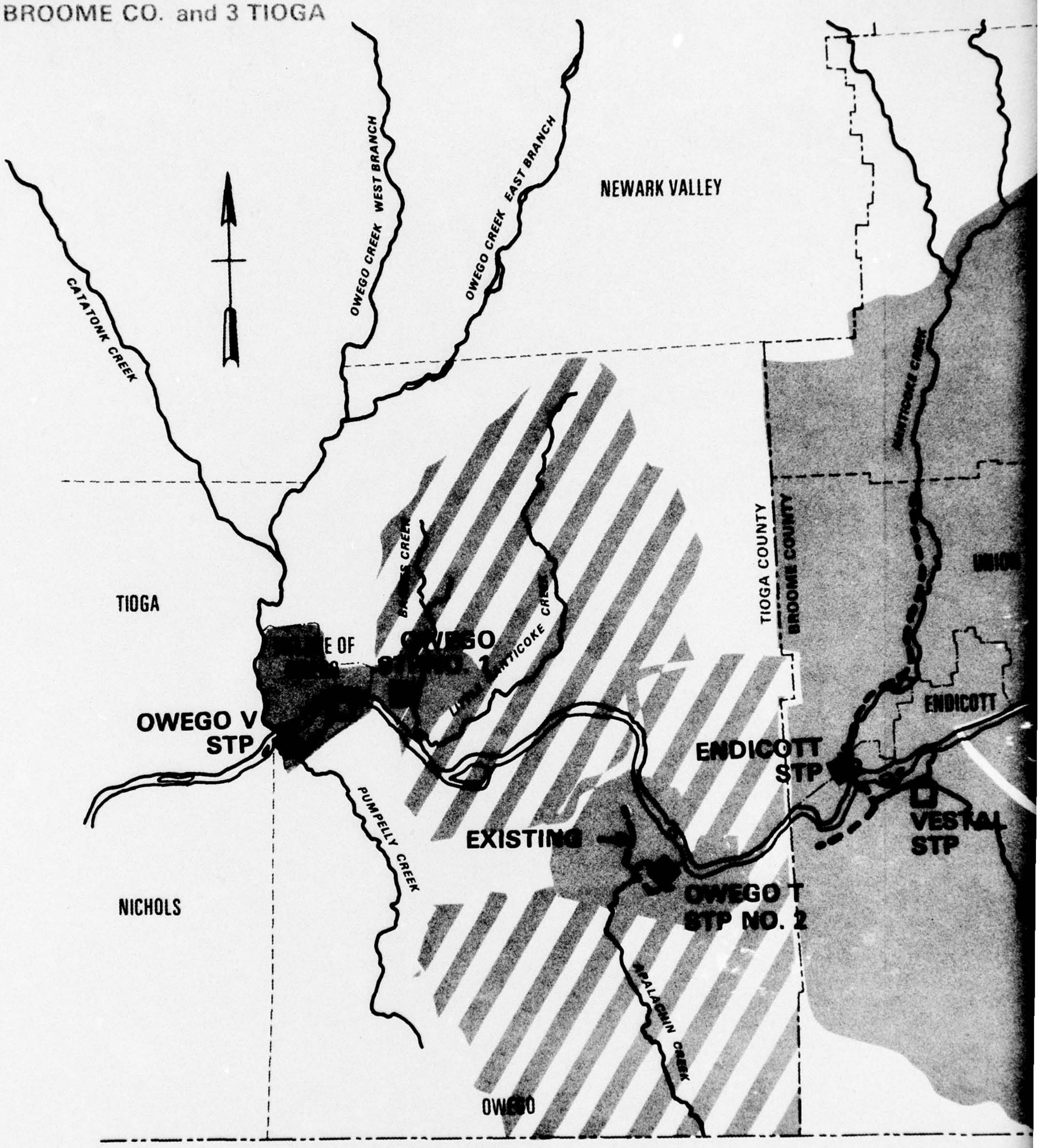
FIVE REGIONAL TREATMENT PLANTS
2 BROOME CO. and 3 TIOGA



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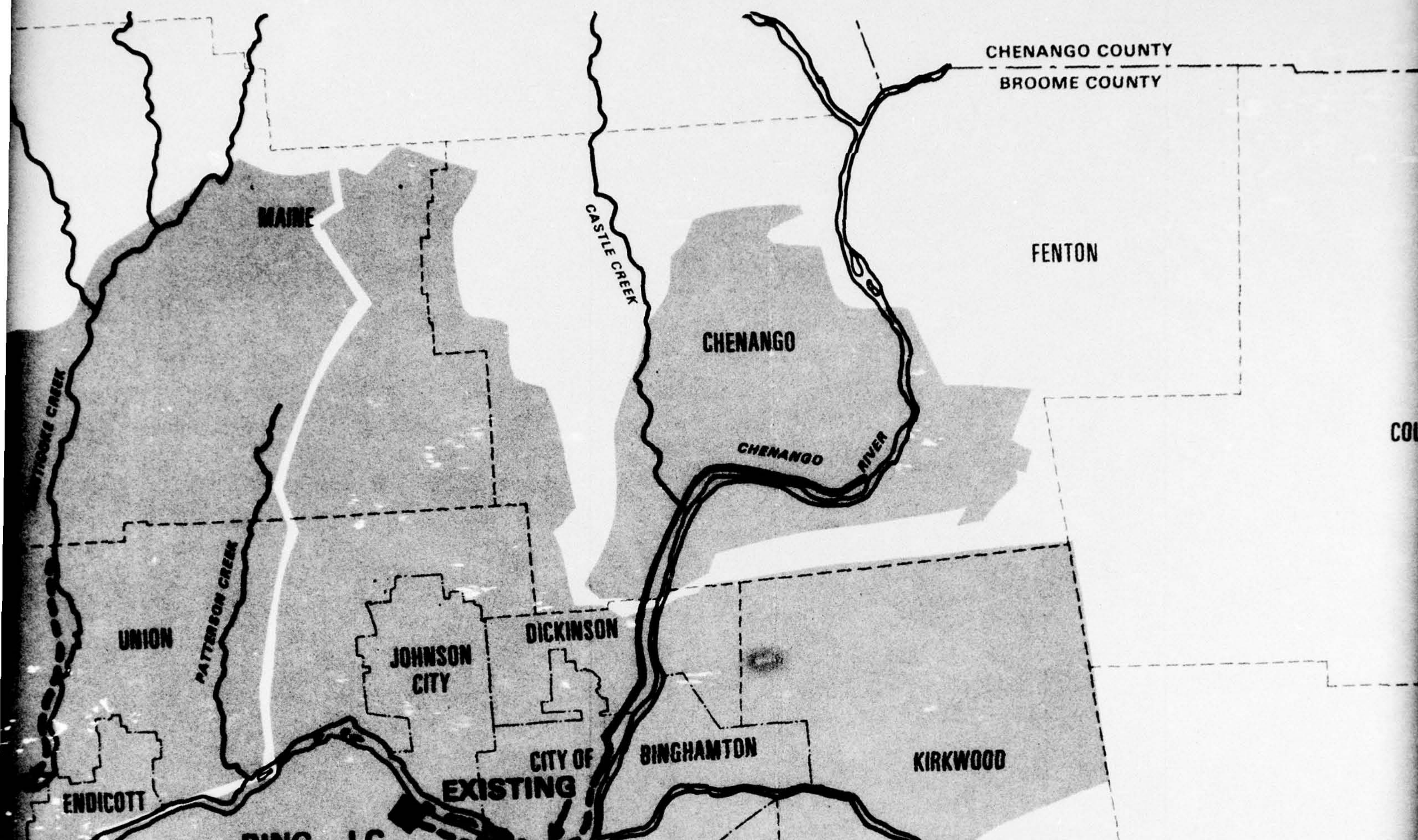

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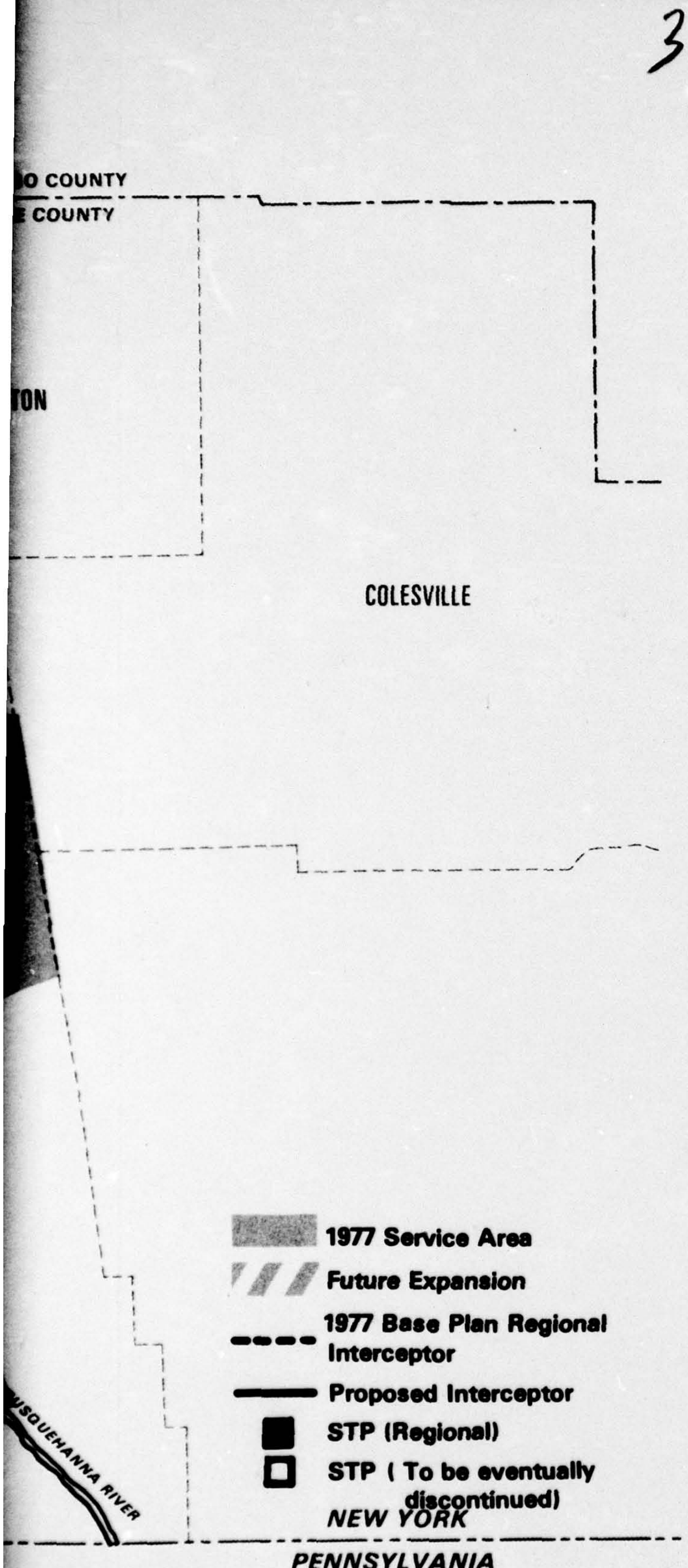
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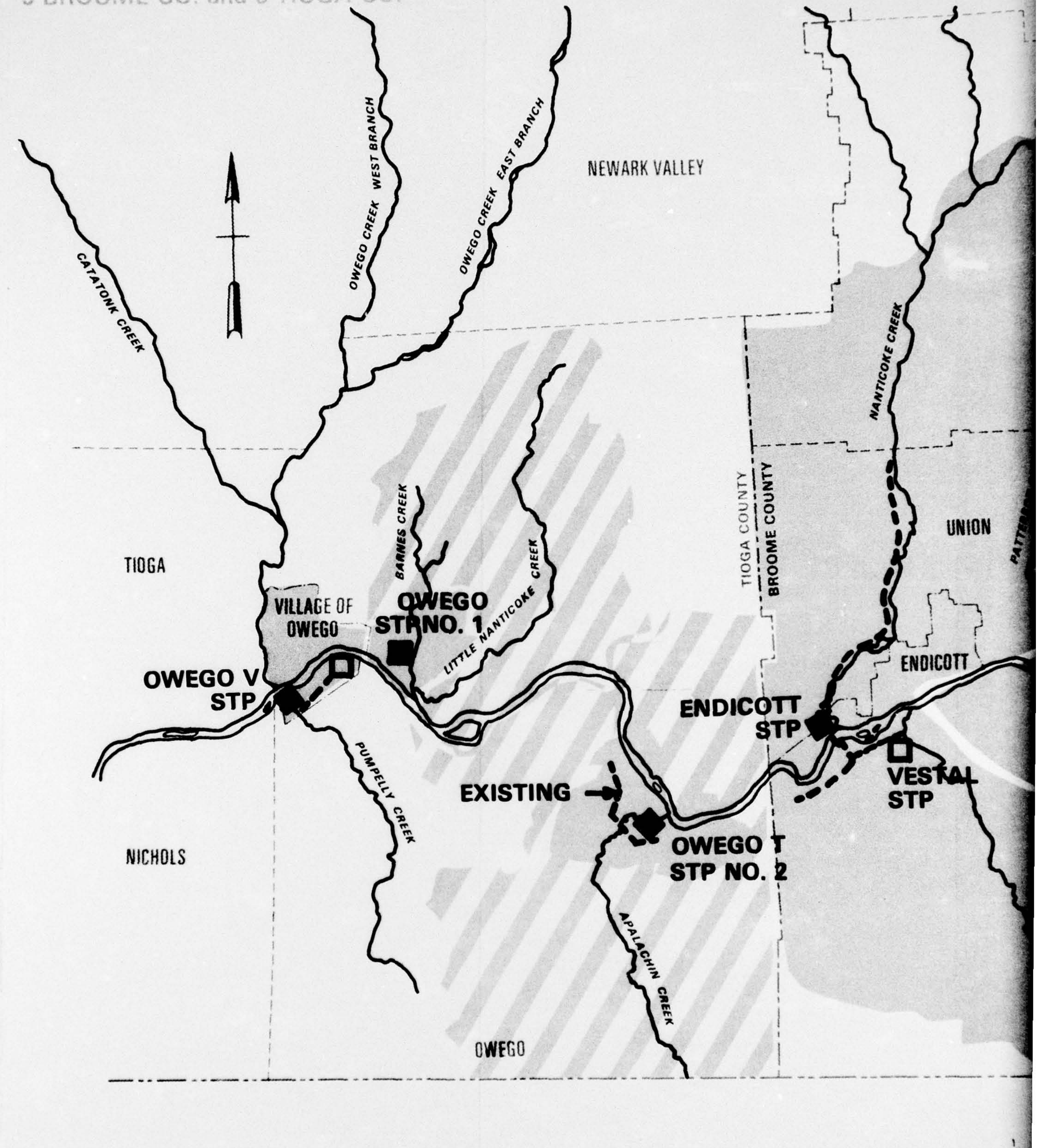
NEW YORK
PENNSYLVANIA

PLATE 4

SIX REGIONAL TREATMENT PLANTS
3 BROOME CO. and 3 TIOGA CO.



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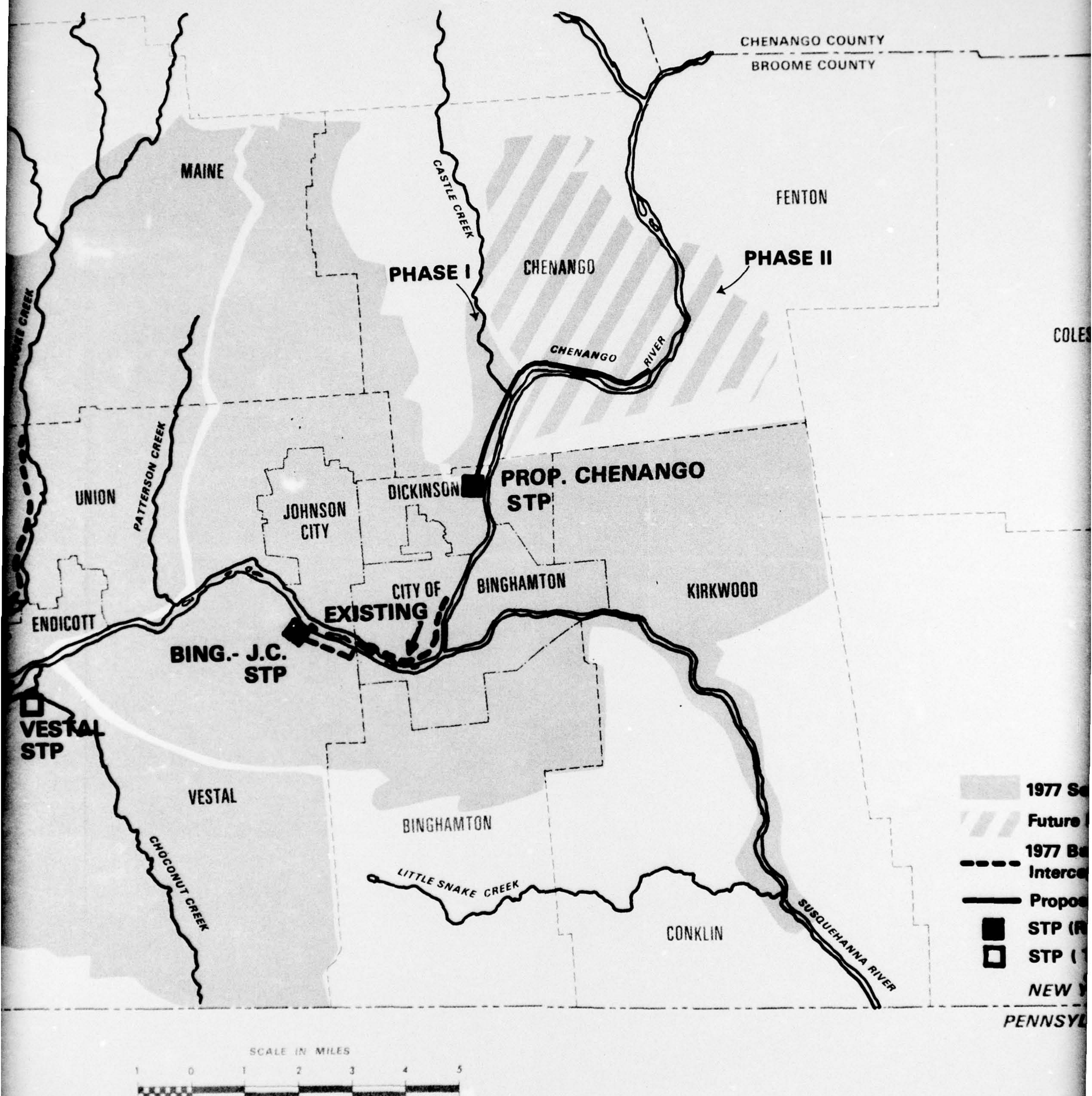

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UNION
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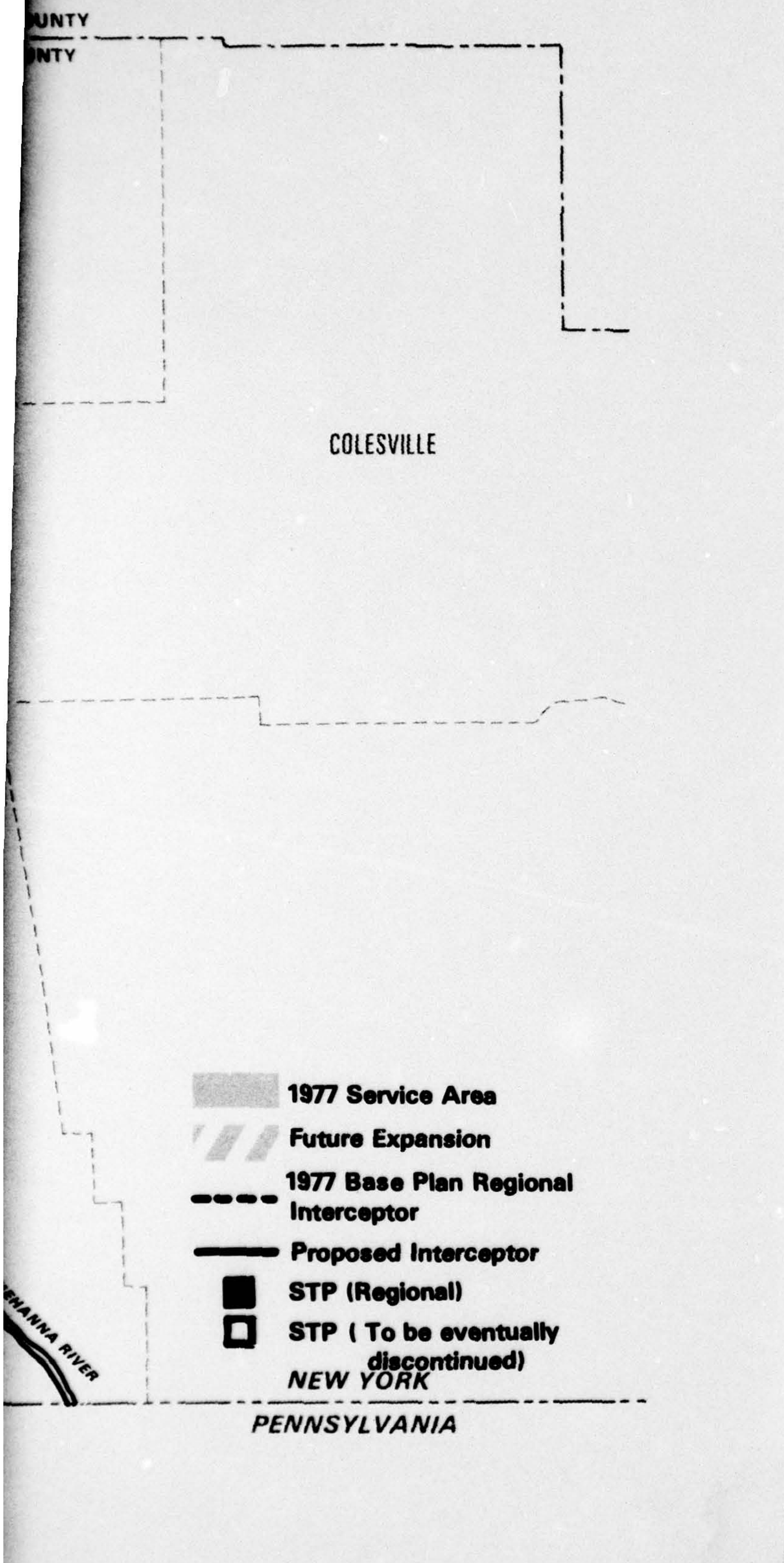
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NEW YORK
PENNSYLVANIA

PLATE 5

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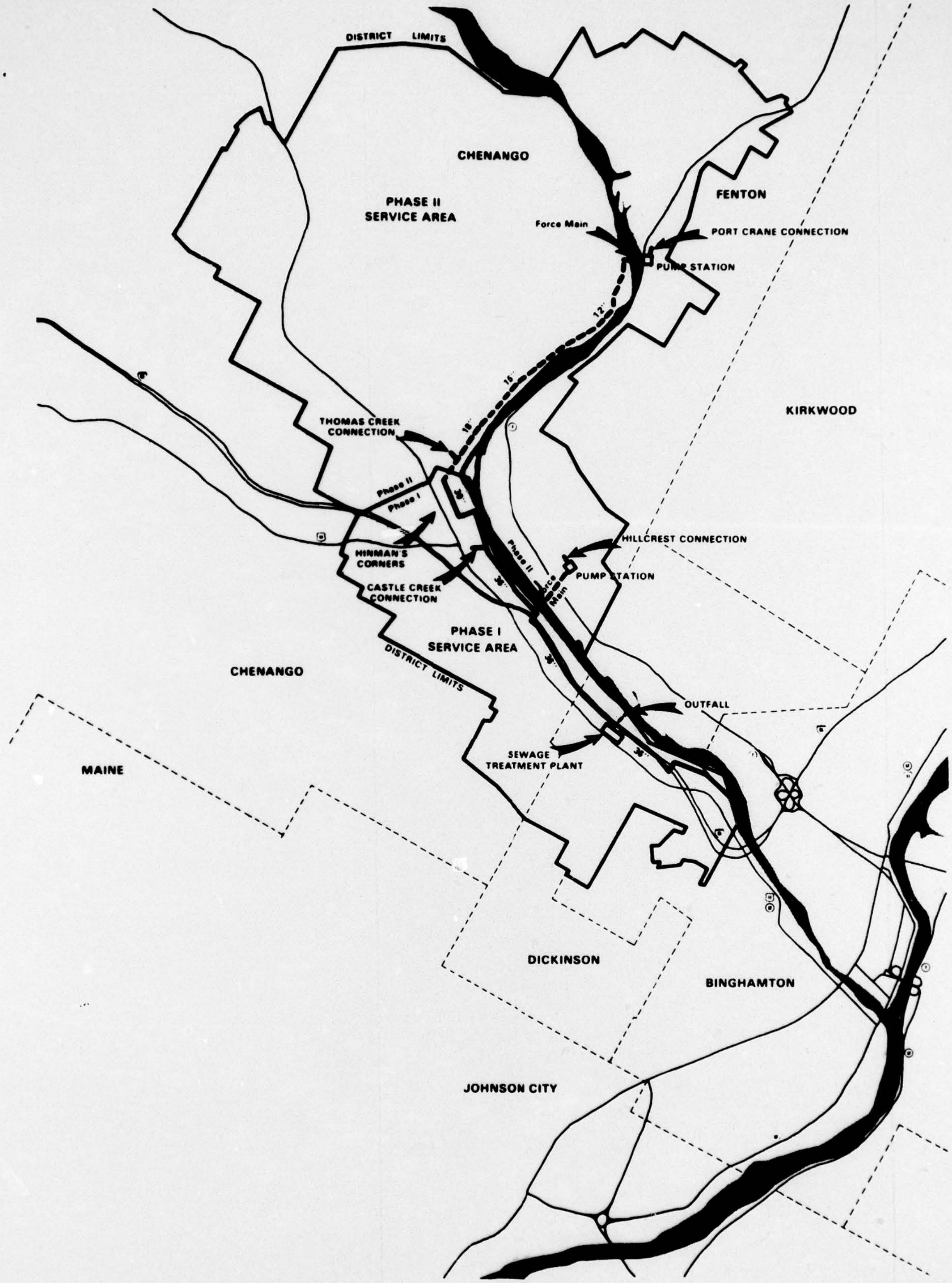

DISTRICT LIMITS
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Force Main
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PUMP STATION
12"
15"
18"
KIRKWOOD
THOMAS CREEK
CONNECTION
Phase II
Phase I
30"
HINMAN'S
CORNERS
CASTLE CREEK
CONNECTION
HILLCREST CONNECTION
Phase II
Force Main
PUMP STATION
PHASE I
SERVICE AREA
DISTRICT LIMITS
CHENANGO
OUTFALL
SEWAGE
TREATMENT PLANT
MAINE
DICKINSON
BINGHAMTON
JOHNSON CITY

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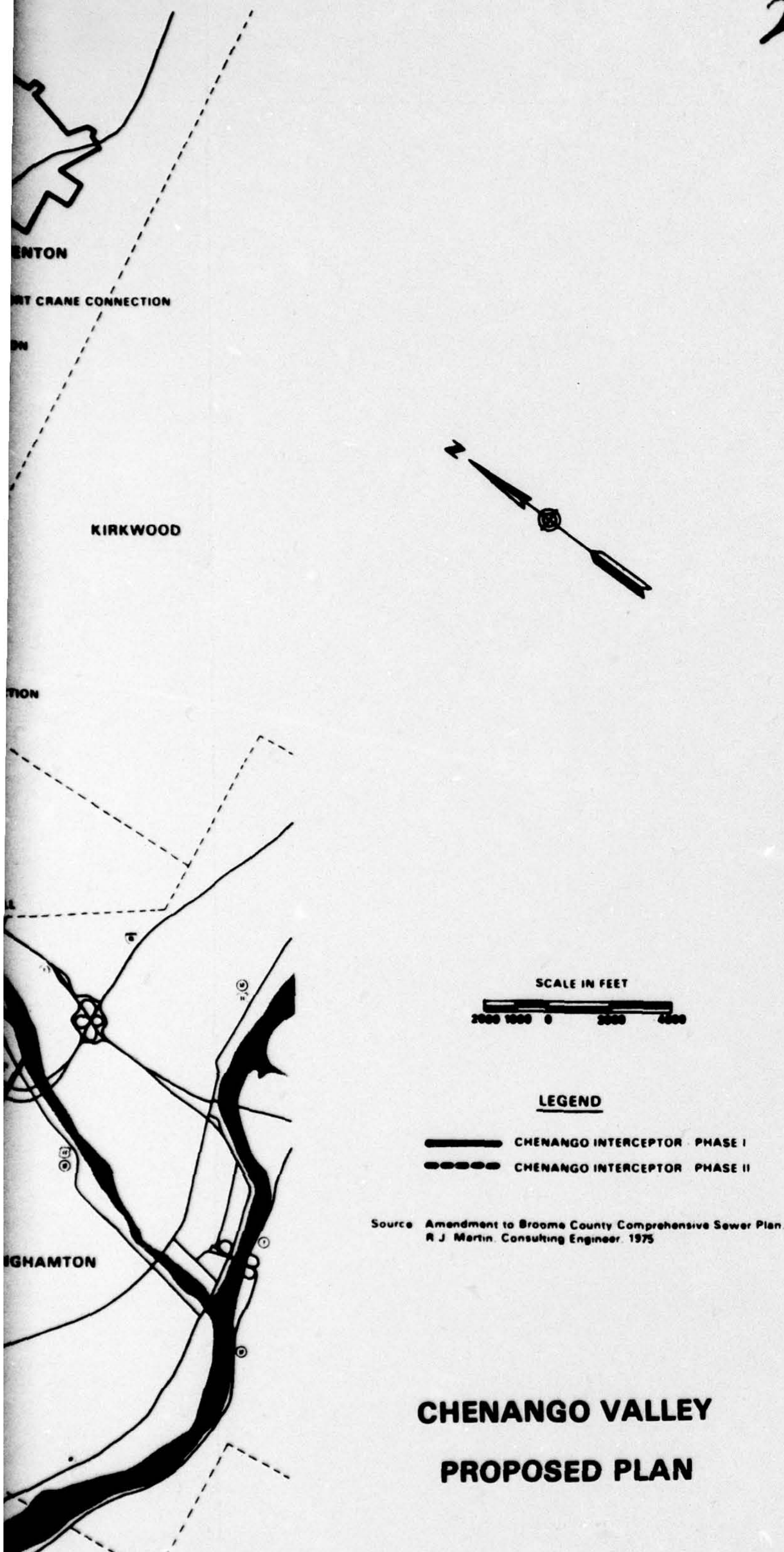

CHENANGO VALLEY

PROPOSED PLAN

PLATE 6